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DISCUSSIONS
APPLICATIONS FOR ADMISSION AND TRANSFER

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

BY ROBERT FLETCHER,[1] M. AM. SOC. C. E., AND J. P. SNOW,[2] M. AM. SOC. C. E.

SYNOPSIS

This story of wooden bridges is written from the engineer's viewpoint. Other writers have presented the æsthetic, non-technical, and sentimental aspect of the subject. Beginning with the dawn of history, the story ends with the evolution of types of timber bridges accomplished only within the two centuries just past; and this development has culminated in the United States, giving to this country the latest types designed in conformity with correct principles and built at minimum cost. In the treatment of the subject three periods may be recognized.

The first period covers what it is now possible to learn concerning wooden bridges constructed by so-called "ancient" people and includes the indefinite period indicated by the historian's phrase, "the fall of the Roman Empire." There seems to be reason for the opinion that types of primitive bridges have survived many centuries in the "unchanging East," and only recently have come distinctly to the knowledge of the Western civilized world. Descriptions of such types are given.

The second period covers the development in Europe from the so-called "Dark Ages" to about the Sixteenth Century, when scientific knowledge began to influence the minds of mankind—the period of the Renaissance. Then, beginning about the middle of the Eighteenth Century, a variety of types appeared, showing much ingenuity in design and skill in construction. It was a time when both rulers and people gave more attention to public works of all descriptions.

Finally, the modern period deals at greater length with the various stages of development through experimental types of American timber bridges to

NOTE.—Discussion on this paper will be closed in February, 1933, *Proceedings*.

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types now accepted as standard. Such are principally those which make use of lattice, Howe, and Pratt forms of truss, with or without a timber arch in combination.

Presumably the end of the period would be when steel and concrete have displaced timber in bridge construction; but it will appear that, where timber is plentiful and considerations of economy and rapid accomplishment prevail, wooden bridges still have "a reason to be." A review of the dominant facts established is given in the "Summary."

Introduction

Extensive inquiries made in twenty-four (only one-half) of the United States (Eastern, Middle, Southern, and Central) have revealed the surprising fact that, as late as 1930, there were between 450 and 500 covered bridges in use, a goodly number of which are likely to serve for some decades to come. A quarto volume, published in 1931, gives 135 illustrations of such bridges.[3] The ultimate settlement of a large area of undeveloped country, such as Alaska, South America, and parts of the Eastern Hemisphere, where timber is abundant, will make demands for bridges that can be built with timber, quickly and cheaply, and that will be durable enough for the pioneer period.

Originally, much of the area of the United States was heavily forested, especially the Eastern part. Naturally the first settlers used the timber at hand for bridges and other construction. The shipyards along the New England Coast, where so many fine ships were built, were effective training schools for heavy timber framing; and the influence of that training still remains among the carpenters of the region. Those conditions led to the building of many bridges of note during the period of development and trade after the Revolutionary War. The spirit of emulation and invention which is characteristic of the American people resulted in a flood of patents for bridge-truss types, a few of which have led to good designs which have been widely approved and adopted. The full text of the investigation upon which this paper was based has been placed on file with Engineering Societies Library, in New York, N. Y., to serve as a possible aid to any subsequent research on this point.

The Ancient Background

Undoubtedly the prehistoric man built his bridges with the adaptable materials within his environment. Where trees abounded, a log fallen or felled so as to bridge a stream, would be a suggestion to the most primitive intelligence. Hence, the simple beam bridge, under favoring conditions, would be a first resort and, from prehistoric time to the present, may be regarded as a world-wide type.

[3] "Covered Bridges in America," by Rosalie Wells, with foreword by Charles S. Whitney, M. Am. Soc. C. E., New York, William Edwin Rudge, 1931.

Again, if the tale[4] of an early explorer in South America may be believed, even the monkey may have taught the prehistoric man. In a tropical forest he may have seen a chain of monkeys, their bodies locked together by means of their long arms and long tails, hanging from the branch of a tall tree and swaying until the lowermost grabbed a branch on the other side of a stream. Then the crossing of the group on this living bridge, the shifting of the point of attachment on that side, the reverse swing of the chain until the rearmost monkey landed, and, finally, the breaking of the chain would be quite an effective demonstration, if and when witnessed.

In the sub-tropical parts of Central Asia, claimed to be the cradle of the human race, bountiful Nature provided ample material for constructing suspension bridges. There is the rattan, or climbing palm, a species of the *genus calamus,* remarkable for the length of its stems growing out from a slender trunk.[5] These stems are very tough, having hooks by which they climb over bushes and trees; sometimes they attain a length of 300 to 800 ft., or more.

Another adaptable material is found in the liana, defined as any perennial plant having woody stems, which are characteristic of certain geographical regions, as the forests of the Amazon, the jungles of India, etc.[6] In the temperate zone the grapevine is an analogous example. The grass rope is a species of bamboo that belongs to the great grass family.

Rope bridges may be classified into five types: (1) Tight rope; (2) single rope and dragline; (3) toboggan; (4) roadway; and (5) tubular type. The tight-rope type, consists of a single rope to serve as a footwalk and two side ropes for hand-rails. This type has been mentioned by the English traveler, E. Kingdon-Ward, to illustrate his article[7] on "Bridge Builders of the Jungle." It is a remarkable fact that this same type of bridge was found by an American explorer in the Cameroon—the French mandate in Equatorial West Africa.[8] It was made safer, however, by a woven net of vines extending from the foot-rope to both hand-ropes. This was among the negro race, so far distant from the Aryan people of the high plateau of Central Asia that it is difficult to understand how, even in the long course of history, there could have been any contact with or communication between them.

In Tibet, rivers are crossed by a single rope which serves, by aid of a trolley block and sling, and a dragline, to transport singly a man, an animal, or a parcel across a stream. (Type (2).)

The toboggan type of bridge (Type (3)) was also described by Mr. Kingdon-Ward.[7] This is a single rope securely anchored to a post or tree higher up than the point of landing on the opposite side of the stream. On this, slides a trolley block from which is hung a sling to support the weight

[4] Described in *Merry's Museum,* a serial magazine for young people, more than sixty years ago. Recollection verified by a contemporary reader. Encyclopedia Britannica, Fourteenth Edition, Vol. 18, Article "Primates," pictures the "spider monkey of tropical America, with long arms and long prehensile tails." These would be capable of the performance described.

[5] Appleton's Cyclopedia, Vol. 13, Article, "Palm."

[6] Webster's International Dictionary.

[7] *Travel,* February, 1931, p. 38.

[8] *National Geographic Magazine,* Vol. 59, February, 1931, p. 231.

of the traveler. The rope and the block are lubricated by yak butter—a product of Asiatic cattle. Standing in the sling, with his hands grasping only the block, the traveler is started by a push, and the force of gravity rushes him across the torrent with fearsome rapidity. Of course, another rope is set for crossing in the opposite direction.

The roadway type (Type (4)), shown in Fig. 1, needs little description. This example is in the Island of Java—Dutch Colonial territory. Apparently, the span is about 100 ft., and the construction is quite like that which an engineer of to-day would make it with the material at hand.

The tubular type (Type (5)), shown in Fig. 2, was another found by Mr. Kingdon-Ward among the fierce Abor people on the Assam frontier. It spans the Dihang River, and is nearly 800 ft. long, the center being 50 ft. above the stream. Report was that 200 men built it in about a month, the labor being freely supplied by the villagers who used it.

A variation of the roadway type of bridge is described by Spanish historians as found by the invaders of the Sixteenth Century in the Peruvian Andes.[9] These bridges were composed of four ropes of twisted vines, covered with branches for a floor, and two ropes for hand-rails. To cross comparatively still water several hundred feet wide, the Inca workmen laid two great ropes of twisted grass on the water surface, on which cross-fascines of dry rushes were lashed. Two other ropes were laid on the fascines, which were again covered with smaller fascines, making a floating bridge with a floor, 15 ft. wide, over which armies passed. Such a structure required renewal or repairs every few months. The Incas also used a single rope on which an under-slung hammock was held by clews so as to be drawn back and forth by a dragline. The widespread use of such "prehistoric" bridges in both hemispheres is significant of antiquity.

A French historian maintains that types of bridges found among backward tribes or communities in the hinterland of more advanced civilization are survivals of prehistoric structures with little if any change.[10] In the hills of Savoy in Italy, a primitive bridge was found, the structure of which had changed very little since the time of Julius Cæsar, who described[11] similar structures used by the Gauls, thus:

"It is a timber bridge or empilage, piled together rudely, not constructed by art. It needs no carpentry. * * * On each bank of the stream a rough foundation of water-worn boulders was laid, about fifteen feet square; upon this a criss-cross (crib-work) of the tree trunks was built so that the logs in the direction of travel, in alternate layers, were made to jut out farther and farther over the water, narrowing the gap to be bridged later by a few logs serving as beams."

Then, a floor of smaller round cuttings laid cross-wise over all made, in modern phrase, a corduroy road. Examples of such bridges in Kashmir, Northern India, and in Bhutan Province, north of Bengal, India, have been published by Charles S. Whitney, M. Am. Soc. C. E. One of these is appar-

[9] Edinburgh Cyclopedia, 1832, Vol. 4, p. 471.

[10] "A Book of Bridges," by Brangwen and Sparrow, Plymouth, England, 1914 (in Dartmouth College Library).

[11] So stated by the author quoted.

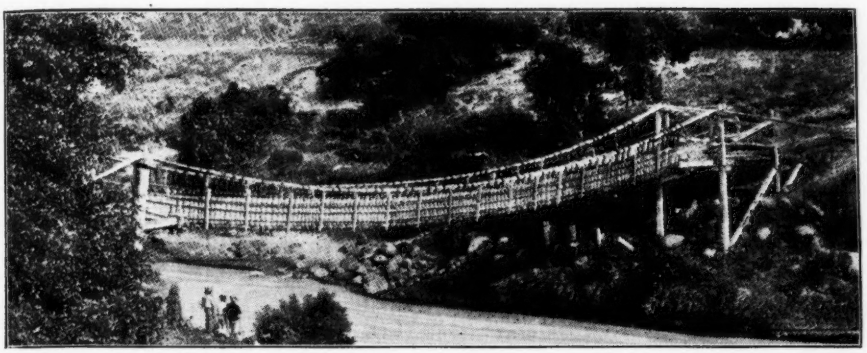

FIG. 1.—HIGHWAY TYPE OF ROPE BRIDGE, JAVA.

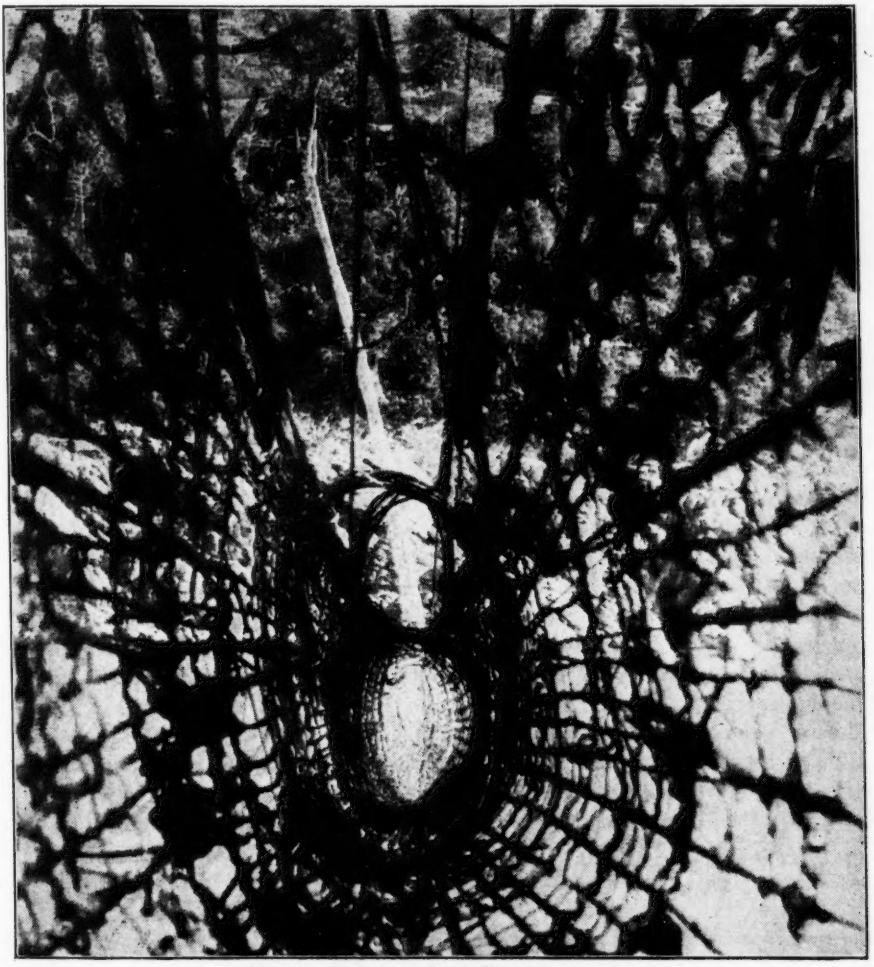

FIG. 2.—TUBULAR CANE BRIDGE, ASSAM.

ently a development of what may be termed the corbel type into a real cantilever type of bridge. The portal towers, which probably serve partly as anchorages, suggest similar portals of Mediæval Europe. The total span appears to be between 80 and 100 ft.[12]

If prehistoric types survive to-day they would be among people like those of the "unchanging East," little affected by time or political disturbance, where the repairs made necessary by the vicissitudes of climate, decay, floods, and destruction by fire or enemies could be quickly and continuously effected.

The early historical period has little to tell about wooden bridges for many centuries. A writer states[13] that the Greeks, although masters of architecture, built few bridges and points out that "even Athens, with its splendid edifices, had no bridge over the Cephissus which ran athwart the most frequented thoroughfare to the city."

As to the Romans, Messrs. Brangwen and Sparrow state[14] that it has not been possible to learn how the Romans built their timber bridges. Pons Sublicious was a sacred monument, hallowed by traditions; but even this bridge was imperfectly described. According to the aforementioned writers, the most that can be said is that the Pons Sublicious was a tree bridge resting on piles. It dated from the time of Ancus Marcius who reigned from 640–616 B. C. Whether the chief priests did or did not build it, they were faithful in keeping it repaired, since it was in existence as late as the reign of Constantine (306 to 337 A. D.)—at least 900 years.

However, a reliable description is available of the timber bridge built (probably) under direction of the distinguished architect and engineer, Vitruvius, about 2 000 years ago, which permitted the passage of the Roman Army into Germany. This is frequently described as Cæsar's bridge. In his day, Palladio, the Venetian architect (1518-1580), made an exhaustive study of the vestiges of the Roman empire.[15] In his treatise on "Architecture," he gives, in the Latin, Cæsar's description of this bridge and a drawing to show the method of construction. The simplicity of the construction is obvious from Fig. 3. Its ingenuity appears by a brief study of the details, which shows how the parts of the bents lock themselves by action of the notched block in the notches on the cross-girder and inclined struts. Imposed weight simply makes the joints tighter. Thus, these bents could be framed quickly and set in place and could be removed quickly without the use of more elaborate fastenings, which would require time and supplies of hardware. It is on record that the Emperor made Vitruvius Superintendent of *Ballistæ* (Chief of Artillery), and this temporary structure in its design gives evidence of such skill as Vitruvius would doubtless possess. It is scarcely probable that the lumber was as smooth and square as the drawing indicates.

A little more than a century later (104 A. D.) Trajan's bridge across the Danube River is mentioned in Roman history as one of the most prodigious

[12] "Whitney's Bridges," by Charles S. Whitney, M. Am. Soc. C. E., Pl. 31, p. 55; Pl. 32, p. 56, Edwin Wm. Rudge, Pub., 1929.

[13] Appleton's Cyclopedia.

[14] "A Book of Bridges," 1914.

[15] "The Architecture of Palladio," Large folio, with many full-page engravings; translated from the Italian, Lond., 1721.

works ever undertaken by the Romans. Dion Cassins stated that it rested on twenty piers, 150 ft. high and 170 ft. apart. These were united by semicircular timber arches. Perhaps the height was fixed by the high banks of the river. Nicholas Bergier[16] states that the piers of squared-dimension stone

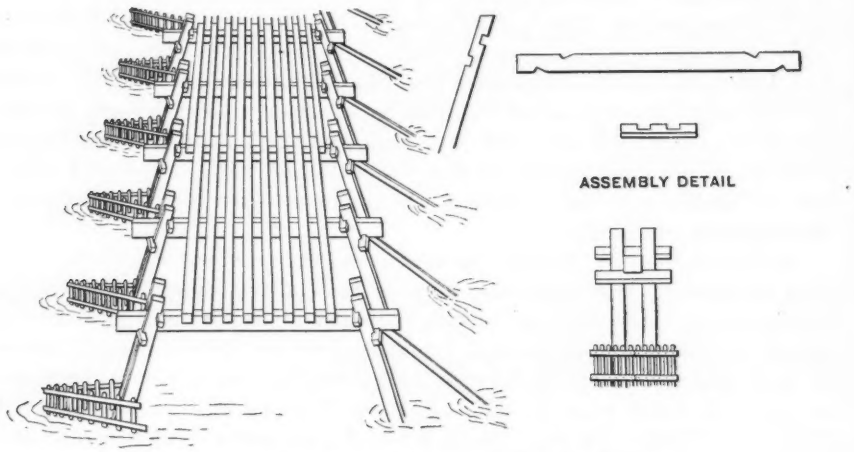

FIG. 3.—CÆSAR'S BRIDGE ACCORDING TO PALLADIO.

were built upon rip-rap dumped upon the unstable bed of the stream and that the legionaries were generally put to work on the roads and bridges to avoid the evils of idleness. Hence, with the customary use of slaves and commandeered labor, the abundance of timber in a wild country, and transportation made easy by the river, work on several piers and timber arches could proceed simultaneously. The carving, showing a span of this bridge, on Trajan's Column in Rome, as illustrated in books, is probably only conventional, since it is not in accord with the description given. Bergier presents long inventories of the roads of the Roman Empire in its greatest extent, from Britain to North Africa, and from Spain to Asia Minor, showing a total of nearly 65 000 miles. As to wooden bridges he states that timber was used in the early stages of road extension, but such bridges were eventually displaced by structures of stone.

LATER EUROPEAN TIMBER BRIDGES

The Sixteenth Century.—The first illustration of a framed truss, in the modern sense, that the writers have yet discovered, appears in Palladio's "Architecture" (Fig. 4). That author states that this truss spanned a mountain torrent in Northern Italy. Apparently, the span is between 50 and 60 ft. The vertical members he terms pillars, or *collonelli.* The peculiar connections for carrying the weight from the cross-girder to the posts are possibly of his "invention," because he describes three "inventions," one of which is similar

[16] "Histoire des Grandes Routes de l'Empire Romain," by Nicholas Bergier, 2 vol.; Vol. 2, livre 4, Chapter 2, p. 610, pub. in Brussels, Belgium, in 1728. Nine chapters on Roman bridges.

to Fig. 4, but with curved chords; a second, which has little merit, and a third, the framed arch shown by Fig. 5.

His designs seem to have attracted little attention because there was no further development of timber bridges in Europe until the middle of the Eighteenth Century—the time of the Renaissance, when both rulers and people gave more attention to public works of all descriptions and when Civil

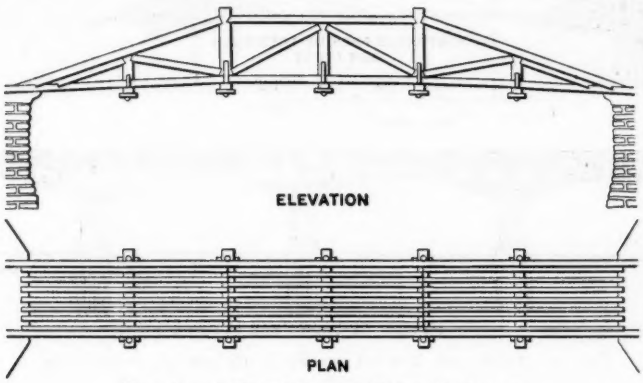

FIG. 4.—A TRUSS DATED ABOUT 1550 A. D.

Engineering began to be recognized as a profession. In this development French engineers took precedence, publishing elaborate treatises on architecture, carpentry, civil constructions, etc. In France, Perronet built his magnificent monumental stone bridges which served as models for English engineers and for those of other lands. He organized "L'Ecole des Ponts et Chaussées," in Paris. English engineers, lacking abundant timber, utilized cast iron in some notable bridges standing to this day; but from the first they followed French practice in constructing some famous stone bridges.

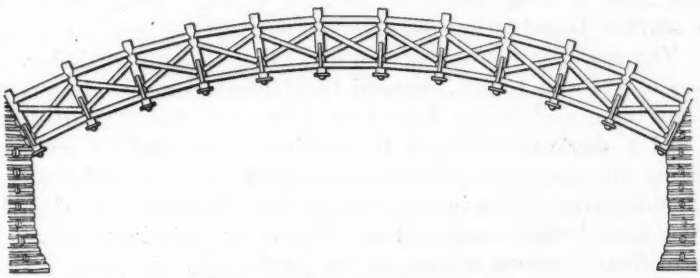

FIG. 5.—PALLADIO'S TIMBER ARCH.

The French types of timber bridges had the distinguishing characteristics shown in Fig. 6;[17] namely, level floors, low parapets, and flat arches, usually not framed after Palladio's pattern, but built of plank in layers clamped together. The floors were supported by radial struts attached to the arch and by braces from piers and abutments. The piers, usually of stone, were

[17] From Gauthey's Atlas accompanying his 3-vol. treatise on "Constructions des Pont."

built at least to the skew-backs of the arches and sometimes to the floor level. The longer span shown (Fig. 6(*a*)) is about 150 ft.; the shorter one (Fig. 6(*b*)), is about 65 ft. In general, the designs produced pleasing archi-

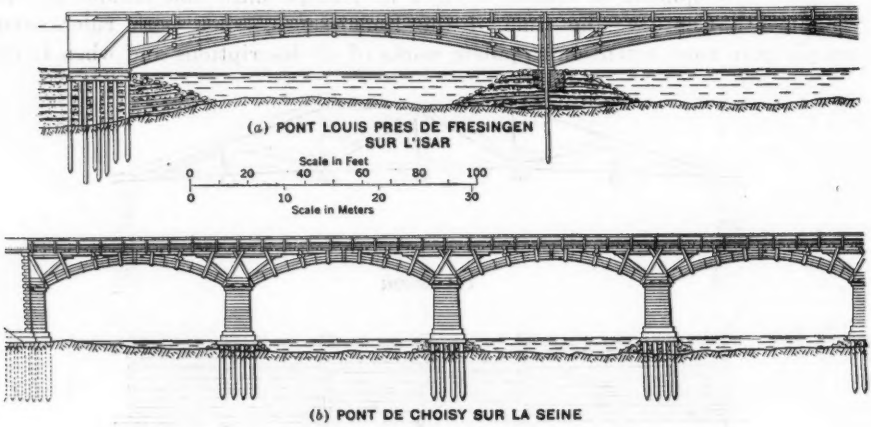

FIG. 6.—EUROPEAN TIMBER BRIDGES, EIGHTEENTH CENTURY.

tectural effects. Occasionally, piers were in the form of pile bents; and simple pile and beam trestles were resorted to when long spans were not needed.

Covered or roofed-in bridges were distinctly not a feature in European practice. Two notable Swiss bridges, built by the Grubenmann Brothers, were exceptional. The picture of the Schaffhausen Bridge has appeared in so many books and atlases as to be familiar to engineers generally. Fig. 7[18] shows the greater part of one span, with the details fairly distinct. It has been overpraised. It was a ponderous combination of long and short braces extending from as many points as possible to both abutments—a maze of timbers, scarfed, bolted, strapped, and clamped together to form nondescript trusses. The roof was top-heavy, with a needless amount of timber. It had two spans of 171 and 193 ft., propped by abutment braces. It was finished in 1758 and destroyed by the French in 1799. The one at Wittingen, Germany,[19] had a single span of 390 ft., making a stretched-out and awkward truss lacking the appearance of adequate strength. Neither bridge was worthy of consideration from an engineering standpoint. The assertion of an English writer that these bridges suggested the idea of the American covered bridge, with its roadway between trusses, is not justified by the facts. The early American types developed in the last two decades of the Eighteenth Century, and later, have little resemblance to those structures.

The only other European roofed bridges discovered[19] are: (1) The "Great Bridge" in Zurich, Switzerland, 128-ft. span, rather clumsily constructed with timbers laboriously scarfed from end to end and bolted together in pairs;

[18] "Notable Bridges"; Atlas of ten large folio engravings pub. between 1799 and 1825, by G. Taylor, 59 High Holborn, London, England.

[19] Gauthey's Atlas, previously mentioned.

(2) a bridge at Bassano, Italy, over the Brenta River, on pile bents in short spans, and covered by an open colonnade supporting a light roof; (3) a project by M. Gauthey for a span of 50 m., which is impractical and not worthy of consideration; and (4) and (5) two other bridges, such as local carpenters might devise. The drawings do not indicate side covering.

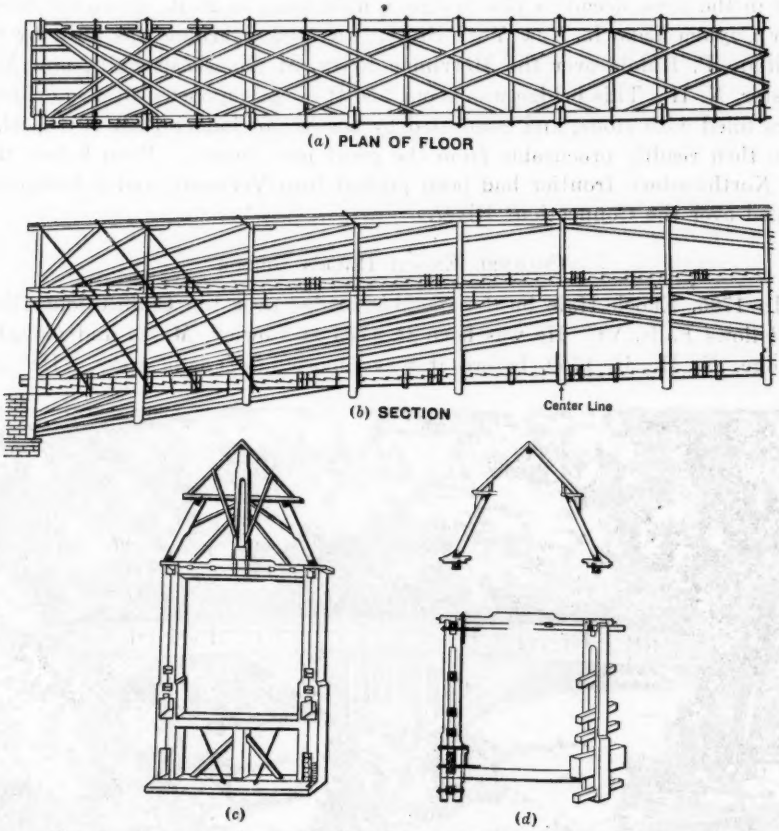

FIG. 7.—PLAN AND SECTIONS, THE SCHAFFHAUSEN BRIDGE, SWITZERLAND.

AMERICAN TIMBER BRIDGES

Development of Types.—The first settlers along the Eastern Coast of America found themselves in comparatively flat country, intersected by broad estuaries and tidal marshes. These conditions sometimes required bridges of considerable length and low elevation which were readily and inexpensively constructed by adopting pile bents and short spans. As loads were light, beams of moderate size were sufficient.

An authoritative history of this development was begun by the late Theodore Cooper, M. Am. Soc. C. E., in a paper presented to the Society more than forty years ago.[20] His short section on wooden bridges begins

[20] "American Railroad Bridges," *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 1.

with the year 1660, thirty years after the settlement of Boston, Mass. He mentions "The Great Bridge" built in 1660, across the Charles River, which was 270 ft. long, on 13 piers. Twenty-five years later, another bridge was built over the same river, 1 503 ft. long, on 75 piers; the West Boston Bridge over the Charles River, 3 500 ft. long, on 180 pile bents, was finished in 1793; and in the same decade a pile bridge, a mile long, in 25-ft. spans, was erected over Cayuga Lake in New York State. In 1792, a bridge was built by Col. William P. Riddle over the Merrimac River, at Amoskeag Falls, now Manchester, N. H. This bridge was about 556 ft. long, supported by five cribwork piers filled with stone, and connected by beams not longer than 40 ft., which were then readily procurable from the great pine forests. Even before that, the Northwestern frontier had been pushed into Vermont, and a bridge was needed over the Connecticut River.

Colonel Enoch Hale's Bridge

In 1785, Enoch Hale built a notable bridge over the Connecticut River, at Bellows Falls, Vt. He was born in 1735 at Rowley, Mass., and moved to Rindge, N. H., in 1760, before it was incorporated as a town. He soon

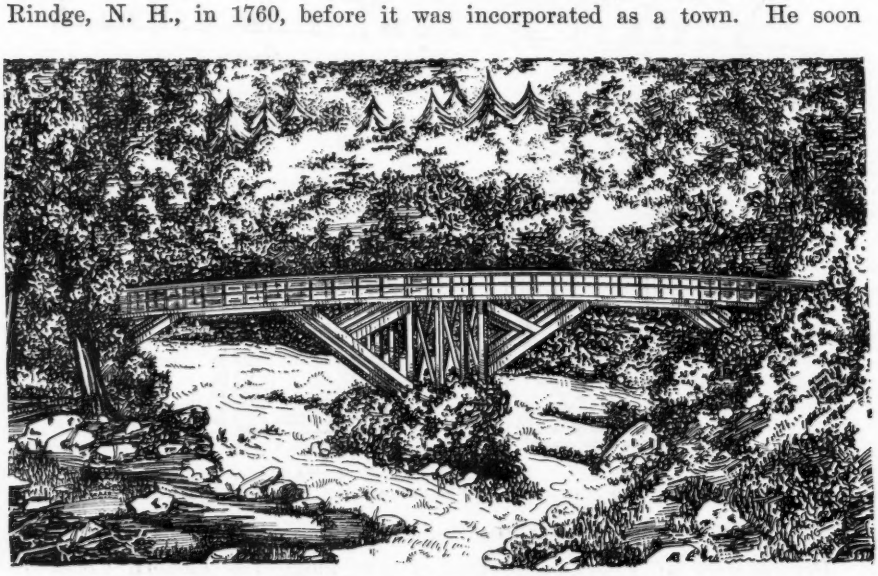

Fig. 8.—Enoch Hale's Bridge, Bellows Falls, Vt.

became prominent in the community, holding finally the office of High Sheriff of Cheshire County. In 1783, he obtained a grant from the Legislature to build a toll bridge over the Connecticut River between Walpole, N. H., and Rockingham, Vt. In 1784, he moved his family to Walpole and completed the bridge in 1785, two years after the close of the Revolutionary War. The bridge crossed the gorge at Bellows Falls, the narrowest point of the river where there is a rock reef dividing the water into two channels. This was the first bridge over the river at any point and while it was being built, the

residents looked upon it as a foolhardy experiment. It is reported by contemporary newspapers to have been 365 ft. long, in two spans, supported at the center on the natural rock pier.

Fig. 8 is a sketch from an oil painting of the locality, showing the original structure, or a successor to it. The date of the painting is uncertain. Undoubtedly, the type shown represents that built by Enoch Hale. It consisted of four lines of braced stringers said to have had spans of 98 ft. each at the foot of the braces. The floor was at least 50 ft. above the water. It was the first bridge built in America with spans greater than could be negotiated with single stringer sticks; and, therefore, richly deserved the fame accorded to it.

The building of this first bridge over the river was noted far and wide. It was considered a remarkable feat of construction. It was reported in many newspapers[21] and was hailed as opening the Vermont region to the trade of Boston and the coast towns of New England generally. It was visited by prominent men. The Rev. Timothy Dwight, President of Yale College, speaks admiringly of it in several of the narratives of his trips up the river. Without doubt the successful building and use of this bridge inspired the building of others in various sections during the next ten or fifteen years.

Timothy Palmer and His Bridges

One of the most ingenious and famous of the bridge builders of this period was Timothy Palmer, of Newburyport, Mass., greatly distinguished as a civil engineer (1751-1821). He may well be called the Nestor of American bridge builders. The principal bridges on the Kennebec, Piscataqua, Merrimac, and Connecticut Rivers were built on his models. Mr. Cooper gives a view of the Essex-Merrimac Bridge, built by Mr. Palmer, three miles above

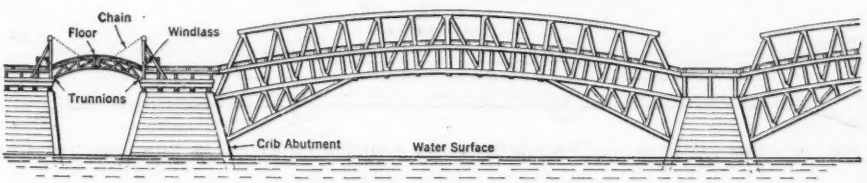

Fig. 9.—Arch Bridge Built at Haverhill, Mass., in 1794.

Newburyport in 1792.[22] It consisted of two arched spans, one on each side of Deer Island. The Piscataqua Bridge, built in 1794, had pile trestle approaches to an arch which was esteemed a wonder at the time, and the Haverhill Bridge, built the same year, has three arches so similar that Fig. 9[23] will serve to illustrate all three bridges. Mr. Cooper, quoting from a contemporary, describes[24] the Piscataqua Bridge ("The Great Arch") as a masterly

[21] *Massachusetts Spy*, February 10, 1785.

[22] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 4, Pl. I.

[23] This was copied from a pen-and-ink sketch made in 1797, now bound with some manuscripts in Boston (Mass.), Public Library.

[24] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 3.

piece of architecture. Its length was 244 ft., the rise was 27 ft. 4 in., and the depth of framework of the arch, 18 ft. 3 in. There were three concentric ribs, the middle one carrying the floor of the bridge. The ribs were made from crooked timbers, so that the fibers were nearly in the direction of the curves, and they were connected by pieces of hard and incompressible wood, with wedges driven between. The ribs were mortised to receive these connecting pieces and wedges, thus keeping an equal and parallel distance between them. Each rib was formed of two pieces, about 15 ft. long, laid side by side in such a manner as to break joints. Their ends all abutted with square joint against each other, and were neither scarfed nor mortised, the two pieces of timber being held together by transverse keys and joints. All the timbers were admirably jointed and freely exposed to the action of the air. Any piece might be removed for replacement without injury to the remainder of the structure. Another account states that, of the three ribs or tiers of girders forming the framed arch, the second or middle one supported the roadway, which was 33 ft. wide, and the third served for a railing.

An inspection of Fig. 9 shows a general conformity to the description of "The Great Arch." The short span of 30 ft. at the left is a draw-bridge over the channel. A feature of these bridges that should be noted, is the long braces at the ends, reaching from points half-way down on the abutments to points three-panel lengths forward on the truss.

Ten years later, in his "Permanent Bridge" (Fig. 10), built over the Schuylkill River (1804-1806), at Philadelphia, Pa., Mr. Palmer changed the

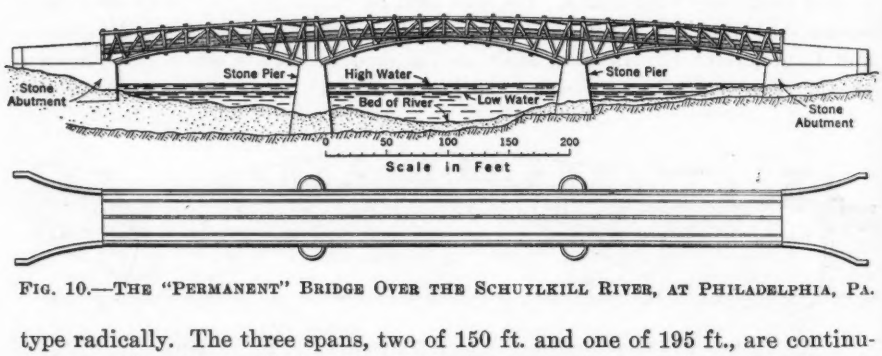

FIG. 10.—THE "PERMANENT" BRIDGE OVER THE SCHUYLKILL RIVER, AT PHILADELPHIA, PA.

type radically. The three spans, two of 150 ft. and one of 195 ft., are continuous over the piers, and there is no bracing beneath the arches. The floor is supported by a heavy continuous string-piece notched into the uprights of the continuous arched truss. The large timber available allowed deep notching and the tension members required projecting ends sufficient to resist shearing. Treenails were used instead of bolts and the timber was the best selected white pine. Meantime, in 1796, Mr. Palmer built a bridge over the Potomac River, at Georgetown, D. C., and secured a patent "for his improvement and construction of bridges." He insisted upon the necessity of protecting bridges from the weather, modestly stating that such precaution might extend the life of the bridge from ten to twelve years to as many as forty years. Whether his were the first of the covered bridges is not known.

His bridge over the Delaware River, at Easton, Pa. (Fig. 11), requires special notice as indicating Mr. Palmer's under-estimate of the durability of his work. The obvious changes from the type of the Schuylkill Bridge are the level floor, the straight upper chord, and the flatter arch. The details to be noted are the slotted bolts, the keys, and wedges. Treenails (commonly

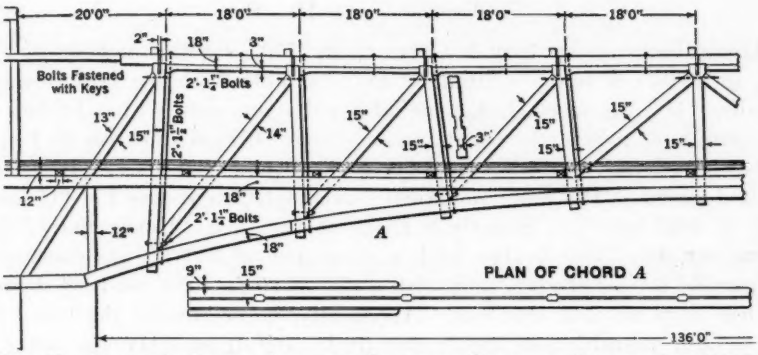

FIG. 11.—PALMER'S BRIDGE OVER THE DELAWARE RIVER, AT EASTON, PA. (1807).

pronounced "trunnels") are in evidence here, also. A recent report[25] states that it was opened as a toll bridge in 1806 and completed a year later, at a total cost of $61 000. It was always covered from the weather, and was removed in 1895, not because of decay, but because it became inadequate for the traffic when horse-cars were replaced by trolley cars. In other words, it was strong enough, after ninety years, for the loads it was designed to sustain.

In Fig. 11, attention should be directed to the style of connections. The bolts at the top of the posts are slotted and held in place by wedge-shaped keys or "fids." These bolts can be driven like drift-bolts in holes of the same size as the bolts, and can be "set up" or drawn in, as the timber shrinks, by re-driving the "fids," or inserting a washer if necessary. Hence, they serve both as treenails and clamps, while a drift-bolt alone serves only as a treenail, and a screw-bolt only as a clamp.

The Second Connecticut River Bridge.—Little more than a decade after Colonel Hale's bridge, a second bridge was built across the Connecticut River which was to be a link on the projected highway connecting Boston with Northern Vermont. It joined the Towns of Hanover, N. H., and Norwich, Vt. Dartmouth College was influential in promoting the project. Professor J. K. Lord is authority for the information that[26] in 1796 one, Rufus Graves, built a bridge consisting of a single arch of 236-ft. span (similar to Fig. 9) at this point. The over-all length of the bridge was 344 ft., its width, 36 ft., and its stone abutments, 40 ft. square. It was arched to such a degree that the roadway at the center of the bridge was about 20 ft. higher than at the ends, presenting thus a sharp ascent on entering and a corresponding down-

[25] Letter from State Highway Dept., Trenton, N. J., March, 1931. Information from Department records, through Roy Mullins, Highway Engr.

[26] "History of Dartmouth College," by J. K. Lord, p. 657.

ward pitch on leaving. It was built of the largest selected pines, 60 ft. long, many of them hewed 18 in. square. The bridge fell by its own weight in 1804. On the testimony given by President Dwight, of Yale College, and others, it seems to be quite probable that Graves' bridge was patterned after the Piscataqua arch, but the builder lacked Palmer's experience and skill.

Louis Wernwag and His Bridges

Louis Wernwag was born in Germany in 1770. He died at Harpers Ferry, Va., in 1843. A letter written by his son, John,[27] to the late Samuel L. Smedley, M. Am. Soc. C. E., includes a list of twenty-nine bridges that Wernwag built during the twenty-seven years of his active career in Pennsylvania, Maryland, Virginia, Kentucky, Ohio, and Delaware. He was an original genius. His most noteworthy accomplishment was his "Colossus," built in 1812 over the Schuylkill River at the then, "Upper Ferry," later, Fairmount, Pa. This bridge, with a clear span of 340 ft. was composed of five parallel arched trusses, each with a rise of 20 ft. The roadway, however, had less than one-half that rise. Apparently, he originated the type. This bold design, scientific and architecturally beautiful, probably was never surpassed in America. One Swiss bridge has been mentioned, with a longer span by 50 ft., but compared to this, it was a monstrosity without merit. The cross-section shows the two roadways and two footwalks.

Wernwag did not take out a patent until 1829 and his drawing[27] shows iron rods in every panel which were intended to act partly as counters and also to adjust the joints if they worked loose. The comparatively light traffic loads of those days would cause but little reaction against the dead weight of such a heavy bridge. The arch was built up in seven layers, and had a total depth of 3 ft. 6 in., and a width of 13 in. Fig. 12 is from the patent drawing and Fig. 13[28] is a full span view of the bridge.

In describing the bridge, Wernwag himself states that dry-rot was prevented by sawing the timbers through the heart to discover defects. The

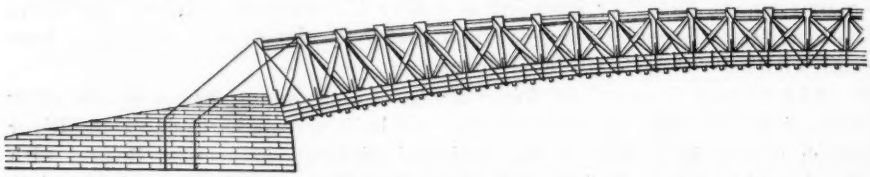

Fig. 12.—Longitudinal Section, The Upper Schuylkill River Bridge, Philadelphia, Pa.; "The Colossus."

timbers were kept apart by iron links and screw-bolts and were neither mortised nor tenoned, except at the king-post and truss ties. No timbers were allowed to come in contact, and provision was made to tighten necessary bolts as the timber tended to shrink, and any piece could be replaced without injuring the superstructure. The trusses were stayed at the ends by two long iron rods, one attached at the top of the end-post, the other at the

[27] *Engineering News*, Vol. 14, (1885), p. 99.

[28] Copied from a large atlas of "Notable Bridges," pub. in London, England, in 1825.

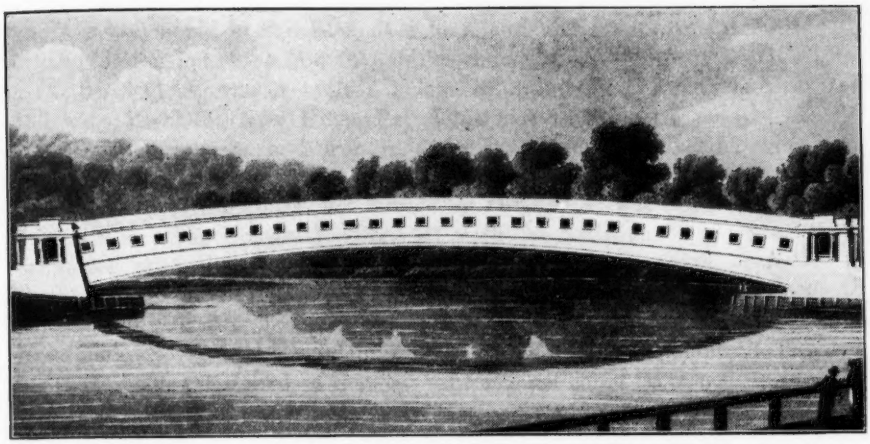

FIG. 13.—THE "COLOSSUS" OVER THE SCHUYLKILL RIVER, AT PHILADELPHIA, PA. (1812).

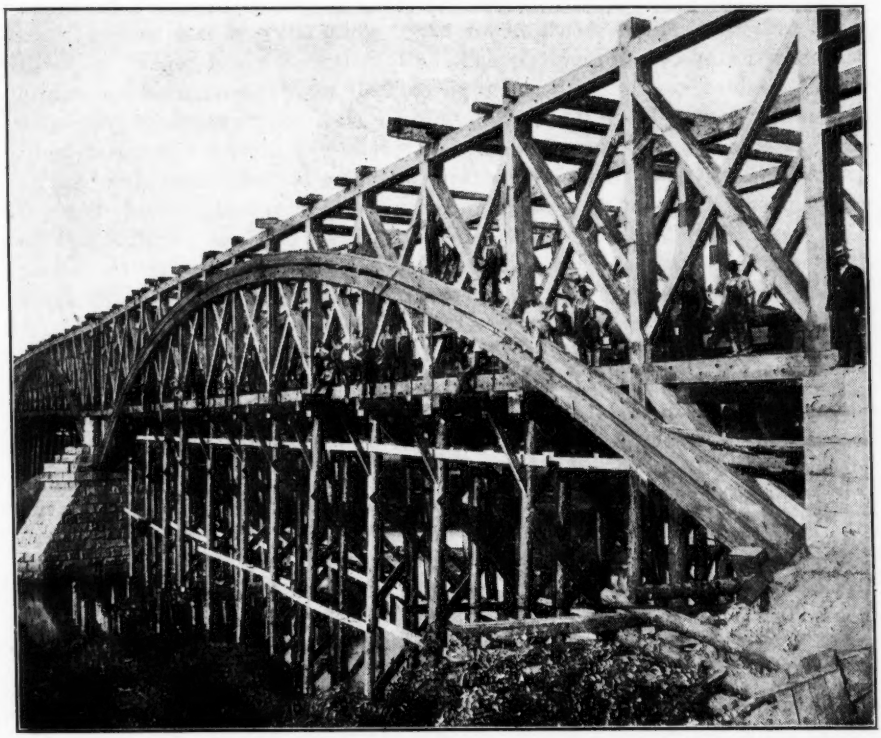

FIG. 14.—BURR BRIDGE OVER THE WHITE RIVER, WHITE RIVER JUNCTION, VT. (1848).

top of the second post. Both extended downward at an angle of about 45° and were anchored at the back of a long and massive abutment built with squared-dimension stone laid in parallel courses.

In 1813-14, Wernwag built a bridge of quite a different type over the Delaware River, at New Hope, Pa. This had trusses with parallel chords, vertical timber posts, and iron rods for diagonals, anticipating what was known later as the Pratt type. The principal feature was the arches which rested on top of the abutments. In other respects, it was based on the same principle as "The Colossus" (Fig. 13) with the addition of the floor chord connected with the casting in which the arch rested, which prevented it from shoving against the pier or abutment.

In 1810 Wernwag constructed a bridge over Frankfort Creek between New York, N. Y., and Philadelphia, which was named "Economy." It was a cantilever[29] type designed so that the center panel could be tipped up in order to pass masted vessels through.

The Burr Truss

Many famous bridges of various designs were built by Theodore Burr, of Massachusetts, during the first two decades of the Nineteenth Century. Fig. 14 illustrates a truss bridge built for railroad service over the White River, at White River Junction, Vt. The photograph was taken when the bridge was being removed in 1890, to be replaced by an iron bridge. It was a splendid example of the Burr type, built in 1848 of old growth white pine when large sticks were available; the end-posts were 18 in. square. The hanger rods were inserted after the bridge was built, to assist in carrying the floor load to the arches as engine loads became heavier. The masonry of this bridge was all quarry-faced granite ashlar laid with very close joints without mortar. At the right end is shown a corner of the north abutment. The abutment wings are circular arcs tangent at the face of the abutment, to a line parallel to the track, presenting the resistance of an arch to the pressure of the earth-fill. The bridge was as strong and serviceable after fifty-four years as when it was built. The entire combination—substructure and superstructure—was capable of much longer service than it rendered. Mr. Burr's first bridge was erected in 1804 over the Hudson River between Waterford and Lansingburgh, N. Y. (Fig. 15). This was followed by a bridge over the Delaware River, at Trenton, N. J., completed in 1806, which carried two roadways and two sidewalks by five arches;[30] the Mohawk River Bridge (1808), at Schenectady, N. Y., which Mr. Cooper described[31] as a wooden suspension bridge; and the long bridge over the Susquehanna River, at Harrisburg, Pa., a wooden arch bridge of twelve spans completed in 1816, on a radically different plan from that of the Trenton Bridge.

These four bridge types seem to have been experimental, in a sense, with Mr. Burr. His patent, dated 1817, which included a drawing of the Water-

[29] *Engineering News*, Vol. XIV, 1885, p. 99.
[30] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), Pl. VII.
[31] *Loc cit.*, Pl. VIII.

ford Bridge type shown in Fig. 15, ultimately claimed nothing but the arch combined with the truss, the arches springing from the face of the abutment below the bridge seat. Although this bridge was built a year or two earlier, it looks as if it might be a derivation and improvement of Palmer's Easton Bridge (Fig. 11). The parallel chords of the trusses made ideal supports for the roof and floor, its posts and other members were firmly connected with the arches at each panel point, thereby acting perfectly as stiffeners for

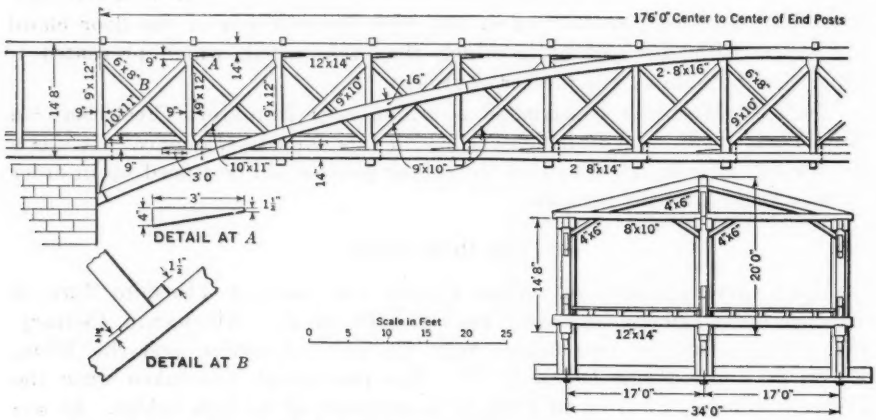

FIG. 15.—HIGHWAY BRIDGE OVER THE HUDSON RIVER BETWEEN WATERFORD AND LANSINGBURGH, N. Y., BUILT BY THEODORE BURR.

the arches. The bridge presents a combination of features the theoretical value of which was little realized by Mr. Burr. The other types, built with splendid workmanship, did commendable service, except the Mohawk River anomaly which had to be held up by a trestle after twenty years of use. The Waterford type, under the name of the Burr truss, was used all over the United States as long as purely wooden trusses were built. The type is well adapted for deck bridges, but few prominent instances of such use have come to the notice of the writers. In this type, Mr. Burr really inaugurated the combined arch and truss which has developed in Twentieth Century steel bridge practice, through the spandril-braced arch into the great braced arches at Hell Gate and Kill van Kull, in New York Harbor, and at Sydney, New South Wales, Australia. What are they but two-ribbed arches with stiffening trusses built between?

The Waterford Bridge, previously mentioned, consisted of four spans of from 154 to 180 ft. clear. It did duty for 105 years without material strengthening until 1909, when it was destroyed by fire. The large timber available and the relatively light loads on bridges in this period of dirt roads rendered these bridges appropriate for the time, although they were frequently designed quite unscientifically. The posts were hewn, not only to make seats for the main diagonals, and for bearings on the arches, but were notched at the lower chord intersections. They acted in tension and had to be framed accordingly. The arches were 4 to 8 in. thick and were attached to each side

of each truss. In case of unusually long spans concentric arches were used. Hundreds of highway bridges framed more or less (generally less), on the Burr principle, were built in various parts of the Eastern and Middle States and in New England. Many of these are still standing. Their longevity is remarkable, if properly protected by roof and side covering. The siding should leave a vertical space at the top about equal to the overhang of the roof; it should be vertical and nailed at the bottom to a girth furred out from the chord to leave a space so that dirt will not collect on the chord. Spans longer than 50 ft. should be reinforced by arches.

The timber in these early bridges was generally white pine which was a factor in their longevity. One at Springfield, Mass., was 104 years old when removed. Two over the Ammonoosuc River, one at Woodsville and the other at Bath, N. H., built in 1828, are still standing (1932). One at Greenfield, Mass., built in 1840, carrying all the traffic from the south, bound for the Mohawk Trail, survived in good condition until June, 1932, giving way then to a new bridge on a revised location. Many of the picturesque old covered bridges still dotting the New England landscape are of the Burr type, most of the shorter ones being without arches.

The longest single span of framed wooden bridge (262 ft.), since Wernwag's "Colossus," of which the writers have knowledge, crossed the Delaware River at Narrowsburg, N. Y. (1859). A system of struts between the posts below the upper chord to resist the horizontal thrust of the main braces, was an unusual feature in a Burr truss.[32] Attention is called to the unusual application of four separate and parallel arches in the two trusses.

The Town Lattice Truss

Ithiel Town, of New Haven, Conn. (1784-1844), was granted a patent on a plank-lattice design for bridge trusses in January, 1820. It represents a lattice web, more suitable for a fence than for a bridge truss. Town secured a patent in 1835 for double webs and secondary chords.

A double-web truss of this type, with secondary chords, is shown in the paper by Mr. Cooper,[33] but it lacks much in the line of bracing and end support for the short lattice. It shows an unusual and unnecessary prolongation of truss over the abutment. It is true that this type requires a wider seat on the masonry than the Howe and Pratt types, in which the entire reaction is concentrated at a single panel point; but the additional masonry required for the lattice type is negligible if the parapet and wings are adjusted properly. Mr. Cooper, and other writers, state that these lattice-truss bridges are unsatisfactory on account of warping and getting out of line. This cannot be wondered at if lower lateral and effective transverse bracing are omitted, as is the case with the drawings usually shown. The scant justice done this type of truss by writers heretofore, is the warrant for recording its merits and according it considerable space in this paper.

Railroad freight traffic in the United States has become so enormously heavy that its economic handling requires equipment so ponderous that

[32] *Engineering News*, Vol. 42, 1899, p. 122.
[33] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), Pl. X.

wooden truss bridges have become obsolete, even on branch lines, because rural industries must transport their supplies and products in the same equipment that is used on the main lines. On account of the common neglect of rural highway bridges it is generally advisable to recommend concrete structures where facilities for assembling materials therefor are reasonable; but where suitable lumber for lattice bridges is abundant and where fabricated steel is costly, as was the case in New England during the Nineteenth Century, and is now the case in Southern Alaska, Northern Russia, and Siberia, and perhaps in some parts of South America, lattice-truss bridges are economical and can be made perfectly good up to spans of 200 ft. for railroad trains not exceeding Cooper's E-40 loading. If properly covered and maintained, such bridges will give from 50 to 100 years' service.

As stated by Mr. Cooper, this truss has been built in many parts of the United States for railroads where suitable timber was plentiful and cheap. It has also been extensively used for highways before and since the advent of railroads in the Valleys of the Connecticut and Merrimac Rivers, in New Hampshire and Vermont. This and modifications of the Burr truss were the favorite types in building the picturesque old covered highway bridges, so many of which dotted the New England landscape and many of which still remain.

In 1900, at least 100 bridges of this type were extant on the Boston and Maine Railroad System.[34] They had been built largely on the "try-and-fail" method by local carpenters who adopted bridge building as their life-work. The best of these bridges, although built without any knowledge of unit strains, were quite satisfactory under the trains then using them. Common defects were lack of effective horizontal and transverse bracing, inadequate splicing of tension chord planks, and unsafe floors under derailed cars. Notwithstanding these defects, due to lack of scientific design, it is remarkable how well the "try-and-fail" method served, in the hands of the naturally competent mechanics of the Nineteenth Century, to produce bridges adapted to the loads and speeds using them. The excellent workmanship of Timothy Palmer's forces in building the "Permanent" bridge (Fig. 10) is typical of the work done by the early builders of these bridges.

Later practice on the Boston and Maine Railroad developed trusses designed more scientifically to overcome the lack of lateral and transverse bracing in the older bridges, as well as to provide for increasing weight of rolling stock. Fig. 16 shows details that have served to keep these structures straight and upright for thirty years and more, although subjected to loads far heavier than those for which they were designed.

The distinctive characteristics of the Town lattice are its use of simple sizes of lumber, the small amount of framing required, and the need of nothing but bolts and a few round rods for metal work. In such regions as those mentioned, far from centers of skilled labor and steel fabrication shops, the simple character of materials and labor required by this type is important. The trusses are generally built of uniform sections throughout. While this

[34] *Journal*, Assoc. of Eng. Societies, Vol. XV, 1895, p. 31.

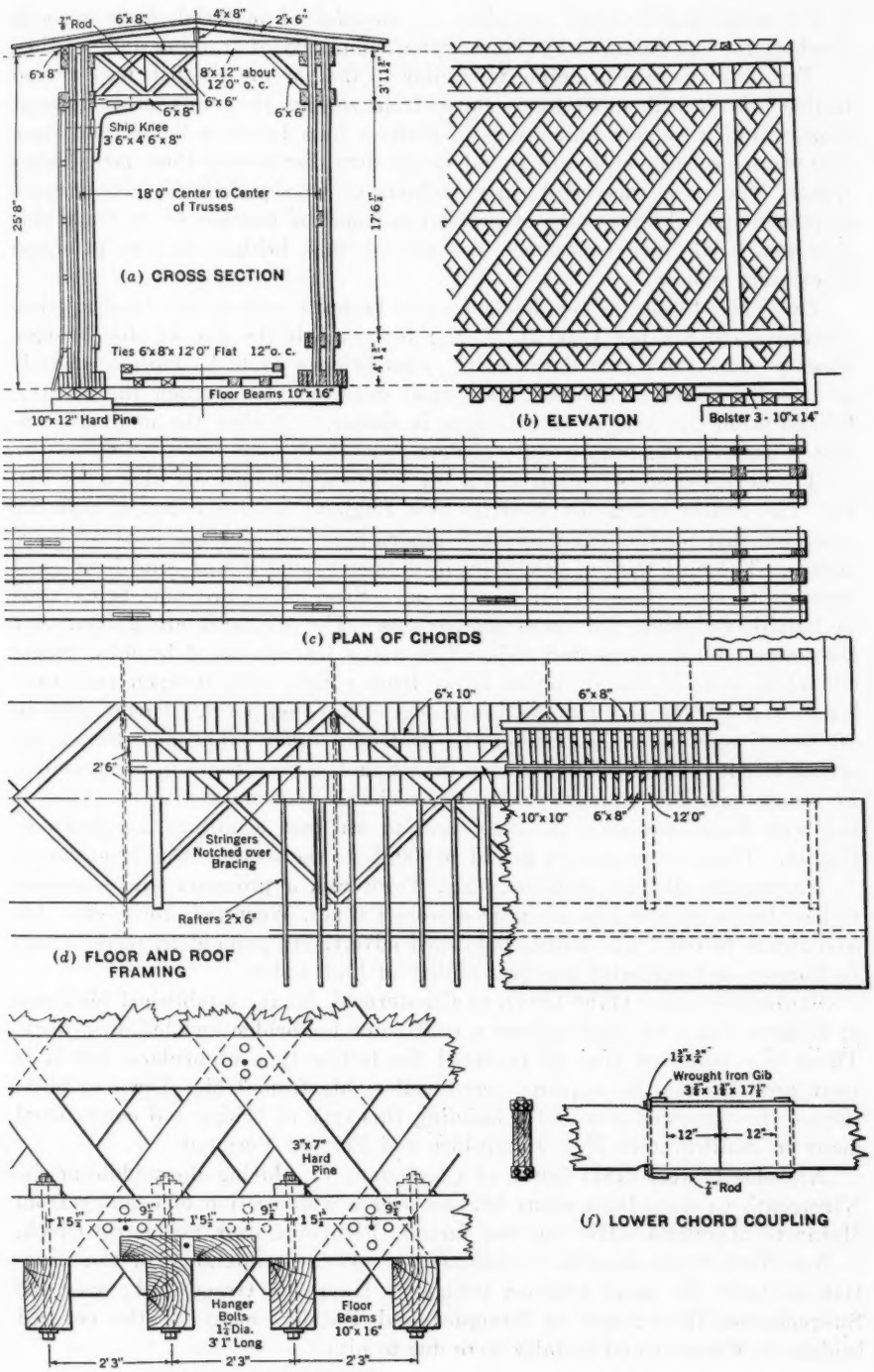

FIG. 16.—TOWN LATTICE TRUSS.

is a feature that is often criticized as wasteful of material, such waste is much more than balanced by the resulting simplicity of framing and erection.

The Town lattice principle is similar to that of the English iron riveted lattice. Both will stand more abuse from service than any other type of truss. Both will give indications of distress long before collapse, and those that were properly built are found doing duty far longer than many other types. The most successful early builders of wooden bridges placed much emphasis upon the necessity of protection from the weather by roof and side covering. All later experience has proved that bridges not so protected deteriorate rapidly.

Until about 1890, lattice bridges could be built with spruce lumber (then costing about $18 per 1 000 ft. B. M.) for one-half the cost of iron bridges. Twenty years after that date (1910), steel bridges could be built as cheaply as those of wood; and now (1931) steel construction is much the cheaper. Computation for designs of this type is simple, following the methods commonly used for computing plate girders.

The accepted construction and a few important details are shown in Fig. 16. The timber ordinarily used in New England has been spruce, with the exception that for the longer spans Southern long-leaf pine was used for lower chords. The pins were of oak, 2 in. in diameter; and it was considered good practice to soak them in lubricating oil. Steel pipes are now being used in British Columbia for these connections. The stringers are gained over the lower lateral braces and rods. The upper laterals are 6 by 6-in. braces and $1\frac{1}{8}$-in. rods in nearly square Howe truss panels, with independent cross-beams and a transverse tenoned brace near each panel point. Turnbuckles in all lateral rods assist in keeping the bridge in order. The floor-beams are spaced to cut the ends of the tension diagonals less than those in compression. To secure sufficient strength in the lower chords of bridges without arches it has been found advisable in some cases to use iron couplings, as shown in Fig. 16. These are computed to add 16 900 lb. to the section if built as shown.

Noteworthy Bridge Builders.—Mr. Town was a promoter and salesman rather than a builder like his predecessors Palmer, Wernwag, Burr, etc. He sold rights to build his design, published advertising pamphlets, made a trip to Europe, and exhibited business ability of high order.

Sanford Granger (1796-1882), of Chesterfield, N. H., established his home at Bellows Falls, Vt., and became a contractor for bridge and building work. There is a tradition that he patented the lattice type of bridges, but it is more probable that he acquired territorial rights from Ithiel Town to build them. He organized a crew for building this type of bridge and constructed many in Southwestern New Hampshire and Eastern Vermont.

Nicholas Powers (1817-1897), of Clarendon, Vt., during the middle of the Nineteenth Century built many bridges over a wide section of country from Maine to Maryland. His was the famous Blenheim, over Schoharie Creek, in New York State, described subsequently. As Superintendent of Construction he built the great railroad bridge at Havre de Grace, Md., over the Susquehanna River; and as Designer and Builder, many of the covered bridges in Vermont and vicinity were due to him.

Dutton Woods (1809-1884), born in Henniker, N. H., adopted bridge building as a life work in 1837 when railroad construction began to be active. He erected many bridges of various types as desired for several railroad companies from New Haven, Conn., to Concord, N. H., and farther north, as well as many highway bridges for various towns and cities. His favorite type in later years was the Town lattice. He used plank arches where needed. He held the position of Superintendent of Bridges on the Concord Railroad until his death.

The Childs Brothers, also of Henniker, N. H.—Horace, Enoch L., and Warren S.—built many bridges throughout New England both for railroads and highways, often in conjunction with Dutton Woods. Horace was the

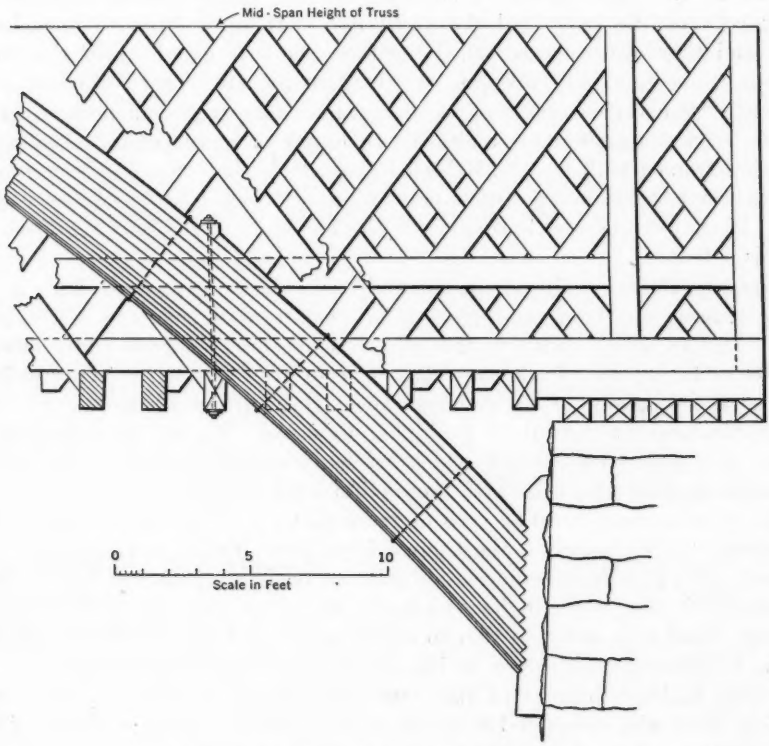

FIG. 17.—TOWN LATTICE TRUSS, WITH BUILT-IN ARCH.

carpenter, Warren the stone mason, and Enoch, the Yale graduate and architect-engineer.

David Haselton, born in Dorchester, N. H., in 1832, deserves mention as a builder of lattice bridges after 1860. He was Superintendent of Bridges and Buildings on the Boston and Lowell Railroad until 1890, when it was leased to the Boston and Maine.

Supplemental Arches.—Spans of more than 80 ft., depending on the load, should be built as combined truss and arch. Ordinarily, a single arch on the

center line of the truss is sufficient, but, if desired, an arch may be built on each side of the truss, Burr fashion. These arches spring from skew-back blocks scribed to fit the masonry below the bridge seat. The arch planks are firmly held together by radial bolts, and the load is applied by vertical suspension rods carrying needle-beams below the lower chords, or by passing through the floor-beams if they are underhung. These suspenders (or posts when over that part of the arch near the end which is below the lower chord) should be not more than 10 ft. apart along the arch to prevent it from buckling on account of its laminated structure. Fig. 17 shows this construction.

Combinations of arch and truss have been disapproved by schoolmen because of the impossibility of determining the theoretical distribution of stress between the truss and the arch; but if there is strength enough in both, and they are properly yoked together, the safe elasticity of the wood and the connections will take care of the distribution. The Town lattice is the best of the various types of wooden trusses to serve as an arch stiffener because its web members serve either as counters or main braces. The value of arches framed with trusses, if heeled against the masonry, has been amply demonstrated by other types of trusses, some of which have stood for a century. If these arches are sprung from the lower chord the purpose of using them is defeated.

Highway Bridges.—Hundreds of highway bridges have been built with lattice trusses, mostly with single lattice; that is, with one web system per truss. Fig. 18 shows such a bridge of two spans built in 1840 and removed in 1930 to be replaced by a two-span concrete arch structure. This bridge replaced the successor of the famous Enoch Hale Bridge (Fig. 8), and was on the identical site, about 15 ft. higher than the old one. It failed from neglect of a decayed wall-block which led to irreparable injury to one truss. It was 262 ft. long over all. Tolls were eliminated in 1904.

The Timber Lattice.—The type of truss shown by Fig. 19 has been used extensively for highways in the Connecticut River Valley and especially in Vermont, and, in a few instances, for railroad bridges. It is not known who first adopted it, but its merits have been abundantly proved. The arches shown below the chord were added in 1927 to relieve a decayed lower chord. (Blenheim Bridge, a different type, shown in Fig. 20, is mentioned subsequently.)

Several highway bridges of this type built 60 or 70 years ago are now standing, some with spans of 160 to 180 ft., over the Connecticut River. The most common reason for unsatisfactory conditions in this type is decay of some part not effectively protected or not renewed in time. In common with plank lattice of their time, built without knowledge of unit strains, they are frequently weak in the tension chord and deplorably deficient in floor-beam strength.

Unlike railroad through bridges, the trusses of highway bridges are not of great height so that they seldom suffer from lack of transverse or lateral bracing. Their plank floors, unlike the open floors of railroad bridges, make the best kind of lateral bracing. The usual construction is a web composed of intersecting joists, 6 by 8 in., or larger, set with broad sides parallel to the

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FIG. 18.—TUCKER BRIDGE, BELLOWS FALLS, VT. (1840).

FIG. 19.—THE LEDYARD BRIDGE, HANOVER, N. H. (1859).

FIG. 20.—BLENHEIM BRIDGE, SCHOHARIE COUNTY, NEW YORK (1855).

plane of the truss, in panels of about 4 ft. which, with trusses of approximately 16 ft., out to out of chords and a run of three panels, make seven crossings of each web member in its length.

At these intersections each member is notched $1\frac{1}{4}$ in., making the web $9\frac{1}{2}$ in. thick. Where the members pass through the chords, both web members and chord-sticks are notched 1 in. each, leaving $7\frac{1}{2}$ in. of net timber for the web and $5\frac{1}{2}$ in. clear between the sections of the chord. For spans of less than 75 ft., secondary chords are not needed. The ends of the diagonals should be cut not less than 9 in. vertically from the chord, in order to allow a safe length to resist the longitudinal shear.

Pony Trusses.—For spans ranging from 30 to 50 ft., pony trusses have been used extensively. Modified Howe trusses with inclined end posts, Town lattice, and mongrel types are common. The trusses of this class must be stayed by sway-braces. It is not good practice to set these braces on an extended floor-beam because each passage of a load would throw the top of the truss back and forth on account of the deflection of the floor-beam. The needle-beams carrying these braces should be blocked down so that the stringers shall not rest on them.

Stringer Bridges.—A history of wooden bridges would not be complete without mention of the humble stringer bridge which was, of course, the earliest recourse of man for crossing deep streams, as stated under "The Ancient Background." In American Colonial days just such structures with log cribs or rough stone abutments and round logs hewn on top to receive a plank floor were used for openings up to 40 ft. For wider streams pile or

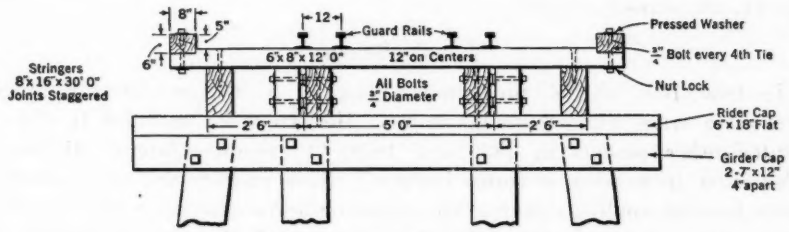

FIG. 21.—STANDARD TRESTLE SECTION.

frame trestle bents were used in quiet waters and log cribs filled with stone where heavy freshets occurred. In the days of large timber and of dirt roads this type sufficed for highways, but with railroads, spans of plain stringer bridges were limited to about 15 ft. Fig. 21 shows a type of single-track, pile-stringer bridge of the last decade of the Nineteenth Century.

For spans from 15 ft. to 30 ft., compound keyed stringers have been found useful. Fig. 22 shows the details. The keys shown are cast iron and are proportioned for the longitudinal shear and, hence, the total depth of the compound stick can be used in computing its moment of resistance. They are placed as nearly as may be in accordance with the intensity of the shear. The thin fin at the middle of the key is cut into the wood to hold the key from working out of place.

Fig. 22 shows the junction of a compound and single stringer on a trestle bent. The pile-cap is called a "girder cap with rider"; it is used only on first-class pile work. For lower grade pile work and for framed bents, solid caps, either tenoned or drift-bolted to the pile or post, are used.

Although it can be said that the day of wooden bridges has passed, it is a fact that many hundreds of miles of wooden trestles, stringer, and truss

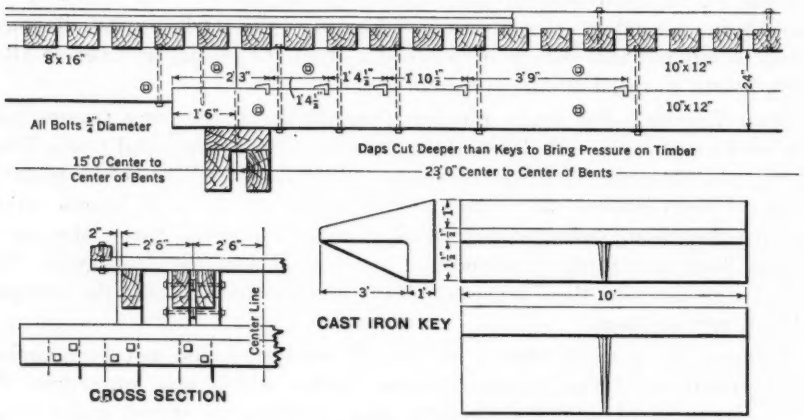

FIG. 22.—KEYED STRINGER SECTIONS.

bridges are still in service in the United States. This brings the discussion down to the simple stringer and trestle, illustrations of which, other than Fig. 21, are scarcely needed.

LONG'S BRIDGES

In 1830, Bvt. Lt.-Col. Stephen H. Long, U. S. Topographical Engineers, patented a truss which bore his name. He afterward modified it and was granted other patents in 1836 and 1839, "to produce lateral stiffness or inflexibility in wooden or frame bridges." This was effected by a system of lattice bracing applied either to the upper or lower chords, or to both. Concerning Colonel Long's truss Mr. Cooper remarks[35] that this is another step in simplicity and concentration of parts. Here also iron did not enter as a necessary part. Connections were made by framing the parts together or by use of wooden keys or treenails. Many bridges of this form were built, but it was never widely popular. Fig. 23, from a drawing of 1830, shows two sets of braces at the abutments below the lower chord reaching forward to the first and second panel points.

The remark of Mr. Cooper applies in particular to the design of 1830. Colonel Long's later designs departed from the simplicity displayed by the first one. The use of abutment braces extending to the first and second panel points breaks the continuity of the frame as a simple truss, independent or unassisted between the bridge-seats. The term, "assisted truss," may be applied to it. The cogged cast-iron splice-blocks (or plates), which his patent

[35] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 9.

drawing shows, might be used instead of wooden splice-blocks. This idea might have found favor in the days of cast iron, so to speak, but is more worthy of consideration now when steel castings with their greater strength are more readily available.

Many rural highway bridges were built throughout the Eastern States by local carpenters, which were hybrids of the Long and Burr types, without

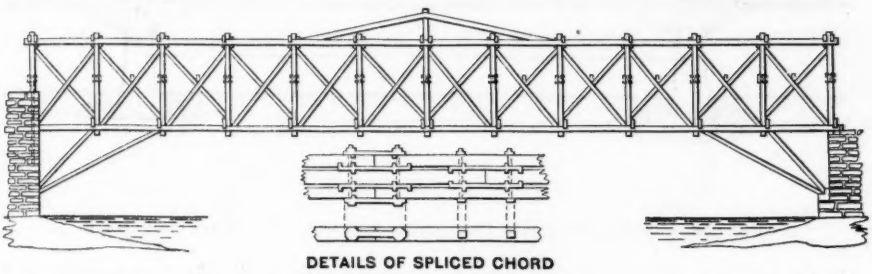

FIG. 23.—LONG'S BRIDGE.

the science of Long nor the arches of Burr. It is impossible to classify them as of any type but, due to the practical common sense of their builders and the accommodating property of wooden framing to distribute strains, these "mongrel" bridges have often given service equal to some of the widely known patented types.

THE HOWE TRUSS

In 1840, a patent was granted to William Howe, of Massachusetts, which was the basis of the Howe truss bridge so generally built throughout the country for half a century and more after the date of the patent. There were two systems of web members; that is, the diagonals ran two panels. The patent drawing also shows a secondary lower chord about 2 ft. or more above the main chord, the only function of which seems to be to carry the horizontal thrust of the end brace of one system of diagonals. It is doubtful whether there was ever a truss built with this arrangement of secondary chord and short end braces. During this season (1840), Mr. Howe, in company with Amasa Stone (1818-1883), built the great bridge over the Connecticut River, at Springfield, Mass., to carry the Western Railroad (later the Boston and Albany), which was opened for traffic the following year. This bridge had seven spans of about 190 ft., center to center of piers. The trusses had two systems of web members like the patent drawing, but the end diagonals of both systems were carried on a single-angle block at the bridge seat. (See Fig. 24.) The lower chord consisted of six sticks, running six panels of about 7 ft. each, packed close together without splices of any kind. The diagonals and rods were of uniform size throughout. In 1874, the bridge was replaced with an iron riveted lattice. It is not known to the writers that there has been another bridge built of the Howe type with webs of two systems. In 1841, Mr. Stone bought the Howe patent, formed a company, and built this type of bridge in many parts of the country.

In this type the vertical iron rods are a vital part of the web system; and the possibility of proportioning the web members for the stresses which they transmit, as well as the simplicity of the joint connections through which that transmission is effected, make this a nearly perfect type of truss.

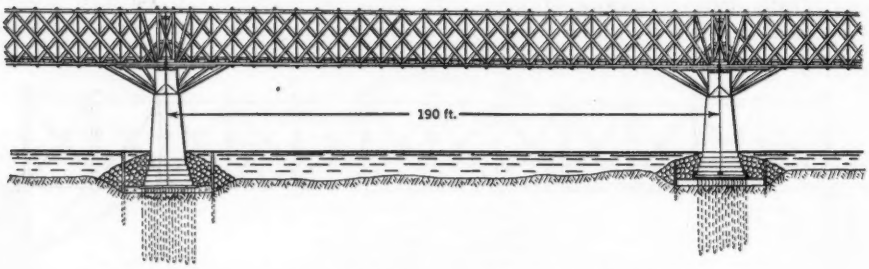

FIG. 24.—BRIDGE OVER THE CONNECTICUT RIVER, AT SPRINGFIELD, MASS.

Early bridges of this type were built of white pine and, later, of spruce, and, finally, of Southern long leaf pine which, although much heavier than the other species named, is better adapted for the Howe type. The splice-clamps and keys are usually of seasoned oak. Oak angle-blocks were used for thirty years or more, but later practice considered cast iron better than wood to transmit the forces acting at the panel point. In general, Fig. 25 shows, by isometric projection, the arrangement of the members of a Howe truss, deck, railroad bridge built in 1874 over the gorge near Quechee, Vt. By cutting off the members assembled at one panel point the view discloses all the truss members and auxiliaries in the bridge, namely, the upper and lower chords, the web system of the main and the counter braces and the vertical rods, the horizontal lateral braces and rods, the vertical sway-braces, the latticed lateral system nearly the same as that proposed by Colonel Long, wooden angle-blocks, and the common arrangement of splice-clamps.

This bridge had a pair of light arches (which is unusual with this type), taking load from the truss by means of vertical rods which, passing through the needle-beams below, received whatever load the latter transferred to it. A proper splice is shown in the lower right-hand chord, and an imperfect, one-side splice is shown on the farther chord at *D*. Each chord-stick was spliced in every fourth panel. This bridge had a clear span of about 200 ft. and a height of 24 ft., center to center of chords. There were eighteen panels, $11\frac{1}{2}$ ft., center to center of verticals. The track was about 165 ft. above Ottaquechee Brook, and the bridge was carefully protected by boarding.

More recent practice improved the application of the Howe design by increasing the size of the web members and decreasing the strength of the lower chord-stick connections from the center of the truss to the ends, according to the stresses sustained. Fig. 26 shows a Howe truss substantially as built in 1889 at Morrisville, Vt., which displays these advances over the older practice. The rods in the panels next to the center are two $1\frac{3}{4}$-in. and one $1\frac{1}{4}$-in. and, in the end panel, three $2\frac{3}{8}$-in. and two $2\frac{1}{4}$-in., all upset, with gib-plates varying from 6 by $1\frac{1}{4}$ by 36 in., next to the center, to 14 by $1\frac{7}{8}$ by

50 in., at the ends. The main braces vary from three sticks, 7 by 7 in., at the center, to three sticks, 11 by 13 in., at the ends. The increase in strength of the lower chord-stick connections from end to center can be seen on the plan. Note the filler opposite the single splice-clamp in the fourth panel from the end. The cast-iron angle blocks are all of the size computed for the end panel. This practice leads to excess of iron, but saves expensive pattern-making.

For through bridges, in order to have sufficiently rigid portal-bracing and to return the side boarding to cover the end panel inside, the upper chords

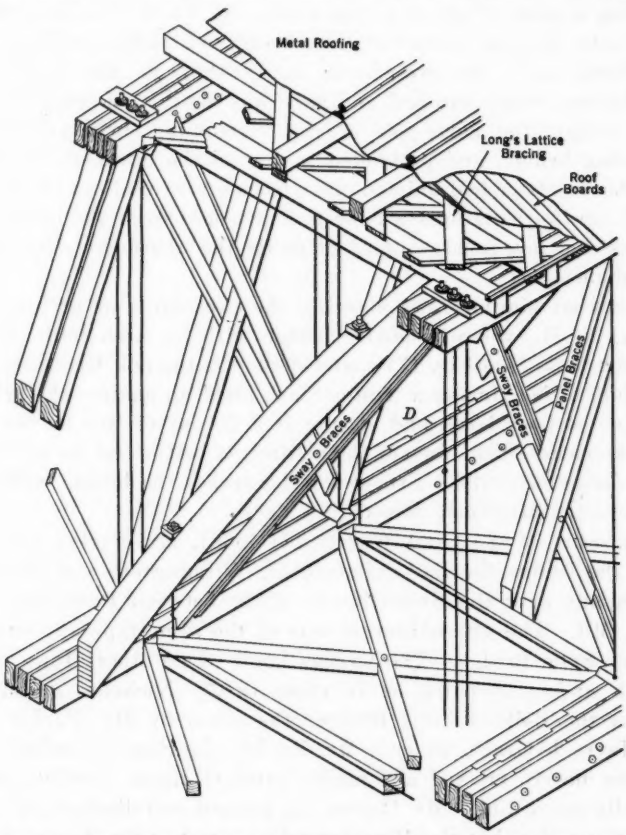

FIG. 25.—ISOMETRIC VIEW OF HOWE TRUSS ACROSS OTTAQUECHEE CREEK, NEAR QUECHEE, VT.

are extended to a connection with the end-posts which may well be in two sets with light rods between them, as shown. Howe truss bridges should be protected from the weather by roofs and side covering, the same as other types of wooden bridges. The lack of this protection has caused the premature failure of many otherwise good bridges, the life of which would have been prolonged two or three times if they had been covered as advised by the early masters of timber framing.

In 1850, a bridge was built for the Sullivan County Railroad Company, at Bellows Falls, Vt., across the Connecticut River, by George A. Parker, of Massachusetts, who, later, was Chief Engineer of the great Havre-de-Grace Bridge over the Susquehanna River in 1862. In the Bellows Falls Bridge were two 250-ft. spans of Howe trusses built with an arch on each side of each truss, Burr fashion. This bridge was replaced in 1882 by a riveted iron bridge on the same masonry that carried the wooden bridge. Arches, although sometimes built with Howe and Pratt trusses, are not indigenous with them so to speak. The south span was saved by its arches during a great freshet in 1869, when a span of the highway bridge at North Charlestown was swept off and came floating down the river erect and whole, striking the railroad bridge "head on." An eye-witness states that, as the lattice-web system of the highway bridge crushed and gave way against the rugged chord of the railroad bridge, the ridge-pole of the "attacking" bridge pierced through the standing bridge, and protruded so far on the down-stream side that, as soon as the wreckage had quieted enough, men ventured out on the bridge and sawed off the overhanging stick for fear it would twist the truss. The bridge was crowded a few inches out of place on the abutment. No further damage was done.

A somewhat similar case, showing the usefulness of arches, occurred at Penacook, N. H., where a lattice bridge with an arch built on the center line of each truss (see Fig. 17) spanned the Contoocook River near its mouth. A great jam of ice, during a winter thaw, piled up against the bridge, several feet above the floor level, and sprung it 2 ft. out of line at the center; but the arches, seated on the face of the abutments, withstood the pressure, and no serious damage occurred. Except for the arches the bridge would have been pushed into the Merrimac River.

During part of the Civil War (until 1866), Mr. Parker was engaged in building the bridge for the Philadelphia, Wilmington, and Baltimore Railroad Company over the Susquehanna River between Perryville and Havre-de-Grace, Md. The superstructure was of the same type and span length as the Bellow Falls Bridge. The writers know of no other Howe truss bridges built with arches in pairs, as in these two.[36] Nicholas Powers, who had worked on the Bellows Falls Bridge, was asked by Mr. Parker to work on that at Havre-de-Grace, under a Mr. Bagley, the Superintendent. When the spans were nearly erected a tornado wrecked them, twisting an approach trestle badly out of line. Mr. Bagley was blamed and discharged. Mr. Powers prepared to go also, but the Manager asked him to take charge of straightening the approach, and finally arranged with him to superintendent the reconstruction. Mr. Powers agreed to do it for $7 per day if, to start with, he could pick fifty men. His terms were agreed to, with the promise of $500 bonus if he finished the bridge without accident. Mr. Powers had his 16-year old son as Assistant, and, at times, had 400 men employed on the work. Meanwhile, the haying season was on at the home farm in Vermont. To the

[36] A good wood engraving of two of the twelve spans of the Havre-de-Grace Bridge resting on stone piers and abutments, may be seen in Appleton's Cyclopedia, Vol. III, p. 273, pub. in 1873.

urgent letters from his wife to come home and secure the hay crop, he replied by sending money to hire the needed help. The bridge was finished sometime in November, a celebration was planned for the dedication, and a picture was to be taken of the bridge crew with Mr. Powers in the center; but he had promised his wife that he and their son would be home for Thanksgiving; he kept the promise and the picture was taken without him.

The great popularity of the Howe truss was well deserved. Many hundreds were built in all sections of the United States and Canada. However, this type cannot be classed strictly with wooden bridges, inasmuch as it uses iron rods for web-tension members—one of the four essential elements of the open framed truss. The development of railroads at the time of Mr. Howe's patent (1840), gave a great impetus to the manufacture of iron, and made it possible to procure round rods in quantity. The type is really intermediate between wooden and iron bridges.

Iron angle-blocks were sometimes cast with tubes extending through the chord, as in Piper's patent of 1867, for the double purpose of furnishing resistance to horizontal thrust by being notched into the chord-sticks; and, also, by contact with the gib-plates, to resist the vertical pull of the rods. Some writers specified that the tubes be made short enough to allow for shrinkage of the chord timbers; but such shrinkage may be more or less than the allowance; and it is best to provide sufficient hold on the chord itself for the horizontal thrust, and also gib-plates large enough to prevent side-grain crushing of the chords. If the tubes and gib-plates come in contact, the chord is generally loose and does not properly support the angle-block. Hence, it is best to omit the tubes and design the angle-blocks and gib-plates to do their part without them. Unless the cast-iron angle blocks are unusually large, it is not profitable to attempt to save metal by coring the central part, although this is the practice of some designers.

Colonel Long's patent of 1830 claims a cast-iron splice plate for the chord-sticks. A modification of this as a steel casting for splicing the chord-sticks in Howe trusses is well worthy of attention. The plate can carry four cogs instead of two, as shown by the wooden splice, and can act as a key by having cogs on both sides and also as a spacer if rests are cast to bring the plate in the center between the spacing of the chord-sticks. The principal difficulty in the use of these plates is in boring the holes if the timber and plates are in place; but it can be done by the skillful use of a ship-auger type of bit (see Fig. 23).

Designing the angle-block of the Howe truss involved the brace-in-panel problem, which is to have the squared ends of the timber brace and the inclined bearing surface of the angle-block fitted so that the center line of the brace and the vertical axis of the rod will meet on the center line of the chord. This involved also the camber to be given to the truss. In theory, it is a problem in higher mathematics. In practice, each builder usually had his own rule, and the best one was to lay out a panel full size and cut a template for the brace, leaving just such an angle-block as he wanted, and then, for camber, cut his counters a little short, the amount determined by his experience. The camber was given partly for the sake of appearance.

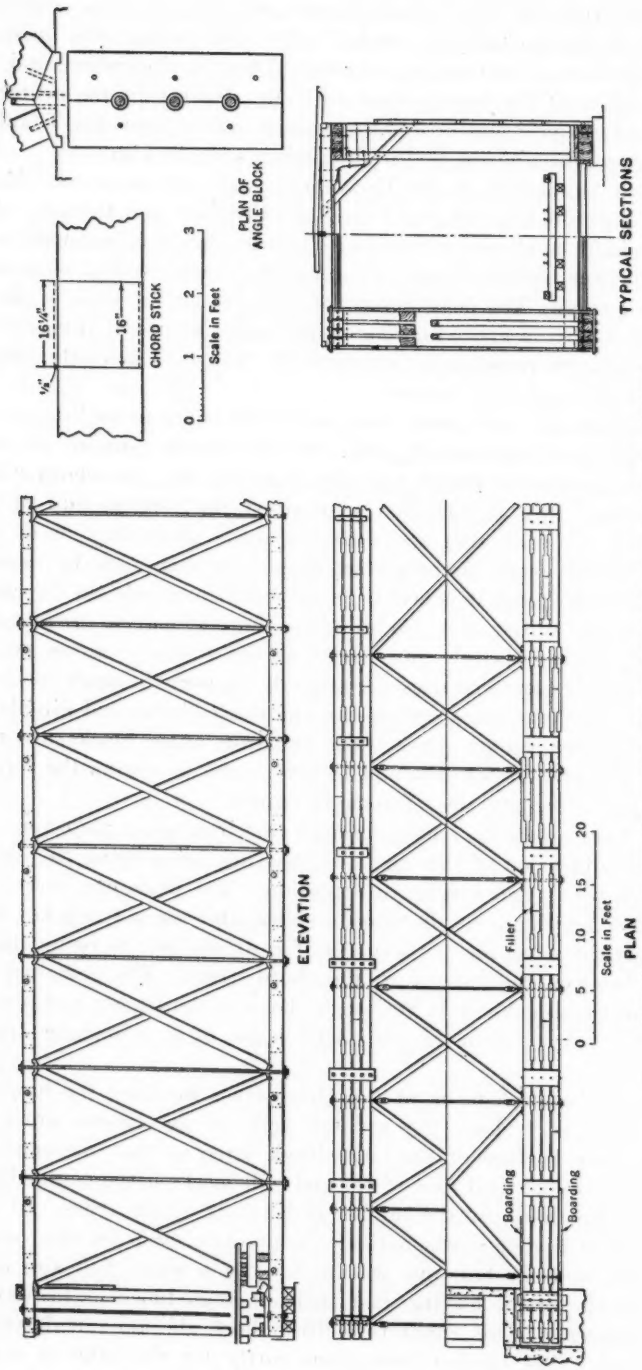

FIG. 26.—PLAN AND SECTIONS, HOWE TRUSS RAILROAD BRIDGE (1870).

Optically, to some observers, a truly horizontal chord might appear to have a slight sag. Even an actual slight sag was of no consequence if it did not increase so as to indicate over-strain of the truss. However, there is a good reason for camber, because of the compressibility of wood and the imperfection of hand-made joints.

Keys and joint clamps should be framed with bearing surfaces and notches cut with a slight draft so as to require driving to place when erected (see details in Fig. 26) and the depth of the notches should be such that the end fiber resistance to the compressive strength at the notch should be equal theoretically to the tensile resistance of the uncut cross-section of the necks of the clamps. The notch must be made far enough back from the end of the chord-stick so that the lengthwise shearing resistance of the softer wood shall be so adjusted to the pull that a slab will not split off between the notch and the end. This is usually the weakest part of the connection. Many splices have failed by such splitting. Almost invariably, in old bridges of this type, outside sticks have only their inner side clamped, as shown at *D* in Fig. 25; and all the clamp bolts are in the head of the clamp, leaving the ends of chord-sticks free to bend outward, which increases the tendency to split off at the notch. The builders were very loath to bore through the neck of the clamps.

The Pratt Truss

A patent was granted in April, 1844, to the late Thomas W. Pratt, M. Am. Soc. C. E., and Caleb Pratt, the claims for which are indicated in a drawing of which Fig. 27 is a copy. In this patent the attempt was made to

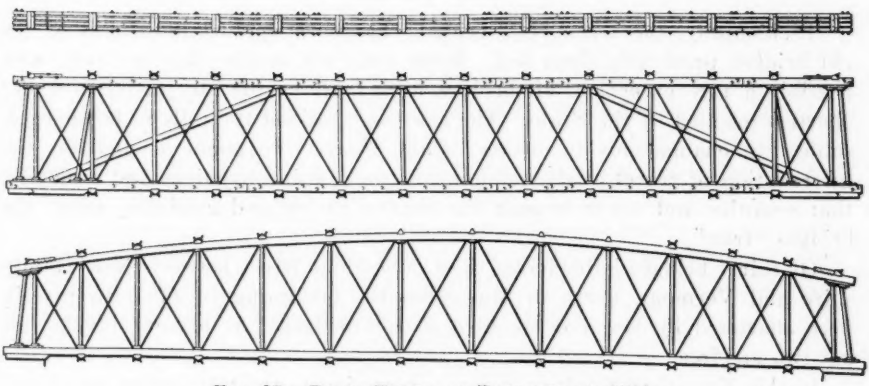

Fig. 27.—Pratt Truss, as Patented in 1844.

use timber in the simplest and most efficient manner in the web members by confining it to the verticals. The advantage of doing this, however, is over-matched by the longer rods required than in the Howe truss, and the awkward angle-blocks needed for these diagonals. For all metal trusses, however, the Pratt truss is ideal.

For railroads, the continuing increase in weight of equipment and loads has kept the problem of strengthening bridges before engineers. For the early Howe and Pratt types it has been found that the tension rods are

the weakest single feature. In the Howe truss, yoking on additional rods is an easy remedy; furthermore, reinforcement is easily applied to the timber diagonals. In the Pratt truss, this is far more difficult; but the Pratt truss permits curved upper chords, as shown in the lower drawing, and this is desirable for long spans. Evidently, in the Howe type, curvature of the upper chord would introduce complications in making proper seats for the varying lengths and inclinations of the diagonals.

In the parallel-chord truss the king brace and strain-beam shown in the upper drawing of Fig. 27 is a reversion to the European practice more than a century old in Pratt's time. In a panel truss it befuddles computation of web stresses and is a faulty device on several counts for assisting a truss which should be designed to be strong enough without it.

The activities of the Childs Brothers in building bridges, especially for the New England railroads when these were young, has been noted herein previously. The drawing accompanying the patent granted to Horace Childs in August, 1846, indicates an attempt to combine the Howe and Pratt types. It is defective in omitting counter-rods in the middle panels, and peculiar because of an awkward attachment of the main braces by a joint bolt passing through the chords and the angle-block, and up into the brace, which was a crude method of making the main braces act as counters. The Childs Brothers built many bridges of various types, among them the Granite Street Bridge, in Manchester, N. H., over the Merrimac River, in 1862, and one over the same river in Hooksett, N. H., in 1858.

Toll Bridges

Incidentally, the term, "toll bridge," has been applied to several of the old bridges previously described. Some may not realize that, at first, with possibly a rare exception, all bridges were built by private initiative under charters granted by the State. The charters specified the tolls to be charged, subject to change only by decree of the Court. In populous districts the interruption of travel at the toll-gates became so inconvenient and annoying that counties and towns bought the charter rights and gradually made the bridges "free."

Of thirty bridges built across the Connecticut River between New Hampshire and Vermont, north of Massachusetts, beginning in 1785, twenty-six were chartered by law and operated as toll-bridges. At present (1932), all but two are free.

In this Twentieth Century, comes a reversion to the toll system, or policy. In the great metropolitan districts, and at points where wide streams must be crossed by trunk-line highways, the cost of erecting and maintaining the huge and lengthy structures, with capacity sufficient for the enormous volumes of light and heavy traffic, is too great to be collected by general taxation. It must be retrieved by toll charges upon those who use the facilities.

Wooden Draw-Bridges

The roads over the broad estuaries and tidal streams mentioned under "American Timber Bridges," invariably required openings for the passage of

water craft. In pioneer days, land transport was difficult, and it was desirable to get water-borne traffic as far inland as high tide would float the small vessels then used. Invariably, draw-bridges called "tip-ups" were built in such cases. They were narrow so that four oak logs sufficed for stringers. Crooked trees were selected with large butts which helped as counterweights. These logs were 12 to 15 ft. longer than the width of the channel to be spanned. The stringer portion over the opening was hewn as small as practicable. An oak log firmly framed to the stringers, with rounded ends to turn as trunnions, served the purpose of a hinge. Chain guys passing over sheaves at the top of a gallows frame spanning the roadway, actuated by hand windlasses, raised the free end of the bridge when desired. If the opening was too wide for a single leaf, two were used meeting at the center, with a toggle arrangement to hold them in grade under passing loads.

These bridges required the lifting of a section of the plank over the hinge, to allow the draw stringers to turn up. This is simple in a highway floor; but with the introduction of railroads, difficulty developed in lifting the rails and their supports, so that a swing bridge of the type called the "**A**-frame swing-draw," was adopted. The stringers of this bridge were framed to rest at the heel—when swinging—on a pivot under the center of the track, at a distance back from the channel cap. The head of the **A**-frame was vertically over the heel pivot when hauled backward for raising the swing end of the bridge. The guy rods running from the head of the frame to the needle-beam under the stringers were held apart for side clearance of the train by a spreader-beam high enough above the track for proper head clearance. A hauling-off beam attached by a pin to the side of the bridge carried a rack into which a pinion, operated by a hand-crank on shore, meshed to swing the bridge off and on,[37] the same as indicated by the illustrations referred to under "The Jackknife Draw-Bridge."

The Jackknife Draw-Bridge.—The **A**-frame draw was awkward to arrange for in some locations. When the Gloucester Branch of the Eastern Railroad was built in 1845, a draw-bridge of some other type was required for the entrance of the little harbor basin at Manchester, Mass. Joseph Ross (1822-1903), a young mechanic of Ipswich, Mass., and Samuel Ashburner, a draftsman, engaged on the work, "got their heads together" and designed what was afterward called the jackknife type of draw-bridge. This important development has been adequately described elsewhere.[38]

A Unique Bridge

A bridge of singular distinction, the pride of Schoharie County, New York, deserves special mention. The location is North Blenheim, on Schoharie Creek. (See Fig. 20.) Erected and long maintained as a toll bridge, it was finally taken over by the town which has carefully kept it in prime condition. The record states that the "contract to design and erect the bridge was awarded to Nick Powers, a Vermont Yankee who built many such structures," and that "the bridge was not built over the stream as many would believe,

[37] *Engineering News-Record*, November 5, 1931, p. 719

[38] *Loc. cit.*, January 31, 1932, Vol. 108, p. 93.

but was constructed piece by piece, back of the present village near the site, and there set up to make certain that every part was perfected before erection over the river was attempted." While this was being done, the masons built the abutments. The material is native oak and pine. The unique feature is the single arch, framed and built on the center line of the middle truss, and this truss extends upward 6 or 7 ft. higher than the side trusses, so that its upper chord forms the ridge of the roof. If the floor timbers extended across the entire 26 ft. of the width, thus serving both roadways, they would contribute greatly to the stiffness and stability of the entire structure.

From available evidence it seems likely that the posts and diagonals were framed to pass through the chords as in Long's design. It has the unusual span of 228 ft., but this gives no distinction over bridges already considered. It can scarcely be classified or correlated with any other type of bridge that has come within the knowledge of the writers. Built in 1855, it is claimed to be quite serviceable for the traffic which it now carries after seventy-six years of usefulness; but the County Officials decided that it must be replaced by a modern concrete structure. However, according to later report, the bridge will be maintained as a local antique.

Retrieving Weak and Injured Bridges

It may be said that it is more feasible to strengthen a wooden bridge in a satisfactory manner than one of metal or masonry; and that the deficiency in the strength of a bridge scientifically designed is more difficult to remedy than in one built by guess, when one member is proportionately weaker than the bridge as a whole. Timber bridges are subject to decay and various accidents, and may develop structural weakness of parts, all of which require timely attention.

First-aid or emergency treatment involves, generally, the setting of trestle bents under the bridge. Where these cannot be permanent, many bridges may be strengthened by the addition of plank arches inside the truss, if side clearance permits. Vertical hanger rods that pass through needle-beams hung below the chords carry the bridge load to the arches.

Lattice bridges are sometimes struck by running ice during spring freshets, and the lattice plank projecting below the lower chord may be broken off, so as to weaken, seriously, the power of the lower chord to carry the floor-beam load. Hanging the lower to the second chord by simple rods or yokes is the usual remedy.

If bridges tend to get out of line, as some do, they are guyed with wire ropes or rods to well-fixed bolts or other permanent anchorage on the banks of the stream. Decayed or broken parts may sometimes be renewed in kind. A common fault in all types is insufficient strength in the floor system. This calls for additional floor-beams or stringers, or both.

Lower chord-stick connections in all types are often deplorably weak in panels near the middle of the span. Fig. 25 at *D* shows a defect exceedingly common. The pull of the single clamp tends to throw the end of the stick

outward, which greatly aggravates the tendency of the head to split at the notch. The remedy for this is an additional clamp cogged in and bolted on the outside; or, if split, by various types of iron couplings shown in Fig. 28, or in Fig. 16, for use in a 4-in. plank. The couplings can be extended to strengthen the central section for a length of, say, 40 ft., in lattice bridges, using loop ends at the gibs and right and left-hand threads for a turnbuckle

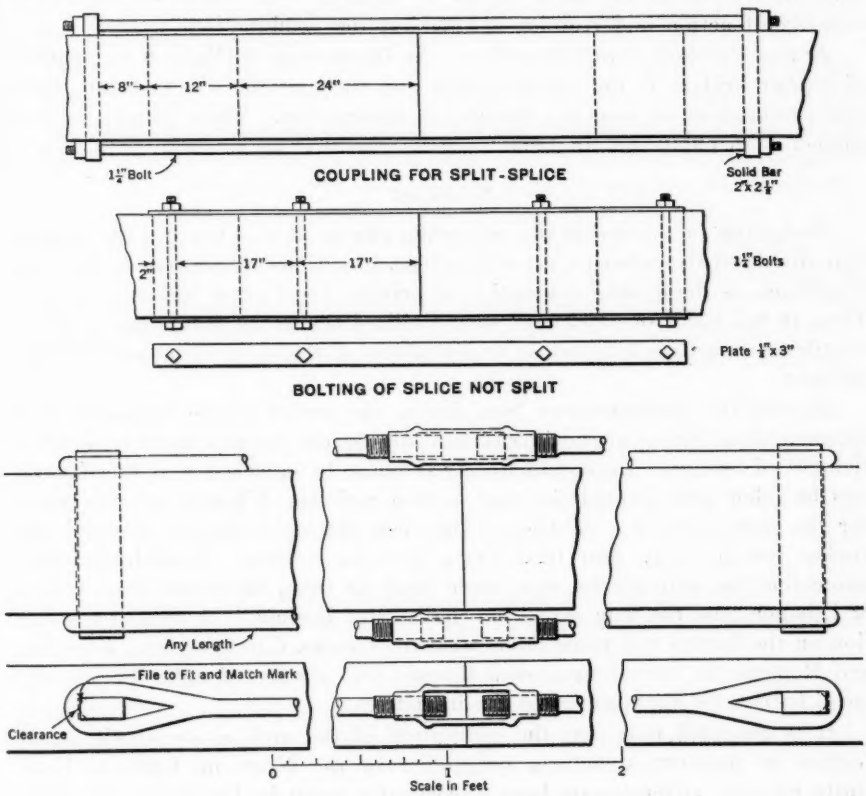

FIG. 28.—REINFORCING COUPLINGS.

at the center. Multiple rods can be used in this manner without trouble. Another favorite method is to attach an auxiliary stick to the side of the chord, connected with effective keys and large bolts or pipe drift-pins.

The tension rods in Howe and Pratt trusses are frequently found to be deficient in strength. Reinforcement is made by yoking a pair of rods to the truss with gib-plates of wood or iron across the chords, as near the panel points as possible. Reinforcement is more difficult in the Pratt than in the Howe trusses on account of fitting the gib-blocks to resist the horizontal pull of the inclined rods. The web-rods in most of the older bridges were not upset, but facilities for upsetting are now such that its neglect in the long rods required for through railroad bridges is inexcusable.

The bridge builder cannot be too careful as to designing the bolted and spliced joints, in order to equate properly the stress and resistance where the parts come together. Manifestly, the tensile strength of coupling bolts and bars, the beam strength of gib-bars and bolts, the compressive resistance of cut-fiber surface and side grain of timber, and the tensile strength of the uncut section of the pieces united or joined, are all factors to be considered. Adjustment should be made so as to secure, with reasonable accuracy, equality of action and reaction throughout the combination.

Bridge Curiosities and Atrocities.—An interesting sidelight of the subject of timber bridges is the example, now and then, of the effect of ignorance and carelessness on essential details of construction. Three letters on this subject were published[39] in 1893.

Summary

Reviewing in succession the advancing phases of this history, the writers have discussed the twisted-vine-cable bridge, in various adaptations to existing conditions, as the natural conception of primitive and early barbarian people. Then, in less barbarous and more semi-civilized times, the overhanging timber cantilever structure followed as an extension of the plain crib-work and log stringer.

Around the Mediterranean Sea, during the period of the colonizing and commercial activities of the Phœnicians and Greeks, there seemed to be little thought of bridges. Land commerce was borne by camels across the deserts and by other pack animals in more settled regions. Wheeled vehicles were, for the most part, war chariots. The then abundant forests provided the timber for the ships sent forth from Tyre and Sidon. Ezekiel, the seer, wrote that the ship planks were made from fir trees, the masts from cedars of Lebanon, and the long oars from the oaks of Bashan.[40] A greater requisition on the forests was made later when the Greeks, Carthaginians, Persians, and Romans, in their long-prowed biremes and tiremes, fought the historic naval battles for the mastery of the inland sea.

It is conceded now that the conception of the arch as an architectural feature in masonry structures originated in the Near, or Farther, East. Quite recently archæologists have uncovered a tomb in Ur of the Chaldees, entered under a true arch of burnt brick. This is said to be 5 000 years old. In Europe, the extensive use of timber for bridges doubtless began when Rome first started to develop its expanding system of military and post roads. Cæsar's Rhine Bridge is a fine type of emergency bridge which would be creditable to a modern engineer. That the early Roman timber bridges embodied the preference for the semi-circular arch appears from the description of Trajan's Danube Bridge. After the long lapse of the Dark Ages, when the Renaissance brought new interest in public works, the flat arch became the standard type. It was expressed in open framing and bracing, such as proposed by Palladio; or solid timber arches, all usually with nearly

[39] *Engineering News*, July 20, 1893, p. 60; September 14, 1893, p. 219; and November 9, 1893, p. 376.

[40] Ezekiel, XXVII, 5 and 6. About the Seventh Century B. C.

level floors and side parapets, with no protection from the weather. The ponderous framing of some Swiss bridges like that at Schaffhausen, with their awkward lengthy bracing, found little acceptance. The low arch types, usually examples of good architecture as well as good engineering, prevailed during the Seventeenth and Eighteenth Centuries, until the later European timber bridges were displaced, during the Eighteenth and Nineteenth Centuries, by the splendid masonry bridges which stand as enduring monuments to distinguished engineers.

Since the aboriginal Americans had little use for products of the forest, the earlier American settlers along the coast had plenty of timber for pile and stringer bridges. Then Enoch Hale made courageous utilization of the magnificent white pine timber at his disposal for bridging the Connecticut River at the Bellows Falls gorge. Following him, the skillful and resourceful Timothy Palmer, and the ingenious and conscientious Lewis Wernwag built their great flat arches of wood somewhat stiffened by truss framework on top. Their insistence on careful protection from the weather inaugurated the distinctly American type of covered "through" bridge. In the first decade of the Nineteenth Centry, Theodore Burr brought out his idea of combining arches springing from the face of the masonry below the bridge seat, with a real truss, the lower chords of which carried the floor. This type of truss perhaps inadvertently on the part of Mr. Burr inaugurated a new era in bridge building, rendering it possible to trace the actions of loads from the point of their application to the supporting masonry.

At the end of the second decade Town brought out the lattice type; and, at the end of the third decade—in 1830—Colonel Long patented his design. These three types, Burr's, Town's, and Long's, are the only important ones, extensively used, that were practically constructed wholly of wood. From 1830, the building of railroads gave a great impetus to the building of bridges; and, in 1840, the Howe type with iron-rod vertical web members, became a favorite; and, in the next decade, the Pratt type, with iron-rod diagonals and timber verticals, soon led to iron lower chords; and then followed the so-called combination bridge interspersed with the early all-iron structures.

It is interesting to note the growth of the use of iron in bridge building. The building of clipper ships along the New England Coast up to the middle of the Nineteenth Century developed there the art of framing heavy timbers, the fastenings of which were made chiefly by treenails ("trunnels"). The iron used was fashioned by the smiths in the shipyards. Timothy Palmer, trained in those yards, used some slotted iron bolts, tightened by keys, as can be detected in Fig. 14. Use of threaded bolts became general afterward. Town used treenails only, but later practice in his type of bridge prescribes bolts freely, and rods where supplementary arches are used. In 1840, Howe made iron-rod verticals the basis of his claim, while Pratt, in 1850, extended their function as diagonals, and also made the curved upper chord a feature. The combination type called for iron diagonals and lower chords with compression members of timber. As early as 1840 all-iron bridges were built with cast-iron compression and wrought-iron tension members. The ingenious and scientific Squire Whipple, Hon. M. Am. Soc. C. E., enters the field here, using

this combination of cast and wrought iron successfully for many years. In 1859, Howard Carroll built the first all wrought-iron bridge for railroad use. It was a riveted lattice truss, and the contest between the riveted and pin-connected types was on. In the last decade of the Nineteenth Century steel took the place of iron as the best material for bridges; and now, ninety years after Howe's introduction of iron as an integral part of the truss, other alloys of steel are coming to replace carbon steel in the art.

Acknowledgments

Special acknowledgment is due to John W. Storrs, M. Am. Soc. C. E., Consulting Engineer, Concord, N. H., for co-operation in research and for valuable information freely given. Others to whom the thanks of the writers are due are, Frank B. Rowell, Research Engineer, Boston and Maine Railroad Company; Lyman S. Hayes, Town Clerk, Rockingham, Vt.; and Mrs. Mary N. Bolles, Bellows Falls, Vt. Several other correspondents have contributed important data without which the history would have been less complete.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

DISTRIBUTION OF SHEAR IN WELDED CONNECTIONS

BY HENRY W. TROELSCH[1], M. AM. SOC. C. E.

SYNOPSIS

The purpose of this paper is to analyze mathematically the distribution of shearing stress along the length of longitudinal fillet welds used for connecting structural members. The analysis will be made only for welds that are continuous over the entire length of a lap joint. In an approximate manner, however, it will apply also to intermittent welds. It can also be used to show the approximate distribution of shear in a line of rivets, provided the deformation or slip of each rivet is assumed to be proportional to its shear.

This paper was prepared in 1929 in an endeavor to interpret theoretically the results of a small number of tests made by the Bureau of Standards in co-operation with the American Bridge Company. Since then several other experimental investigations dealing with the same subject have been undertaken, and it is felt that the mathematical analysis will be of sufficient interest to warrant its publication.

Assume that two members, carrying either tension or compression, each made up of one or more bars of equal section, are spliced by lapping the bars of one member over those of the other in a symmetrical arrangement, and that they are joined by longitudinal fillet welds extending the full length of the lap. Such a splice is shown in Fig. 1.

Fig. 1(*c*) is a diagrammatic representation of this joint, in which, for simplicity in analysis, each member is shown as a single bar. It will be assumed that at any section, *f-f*, the stress in each bar is distributed uniformly over that bar, and that there is no tension or compression in the weld metal. The welds, as well as the metal of the bars, will be assumed to deform

NOTE.—Discussion on this paper will be closed in **February, 1933**, *Proceedings*.

[1] Designing Engr., Am. Bridge Co., New York, N. Y.

according to Hooke's law. The shearing deformation of the weld at any point will then be proportional to the shearing stress per unit of length at that point. This may be expressed by the formula,

$$q = \frac{v}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, q is the shearing deformation of the weld; v, the shear per unit of length of weld; and D, the "detrusion ratio," or the measure of the stiffness of the weld. The value of D depends on the nature of the weld, and must

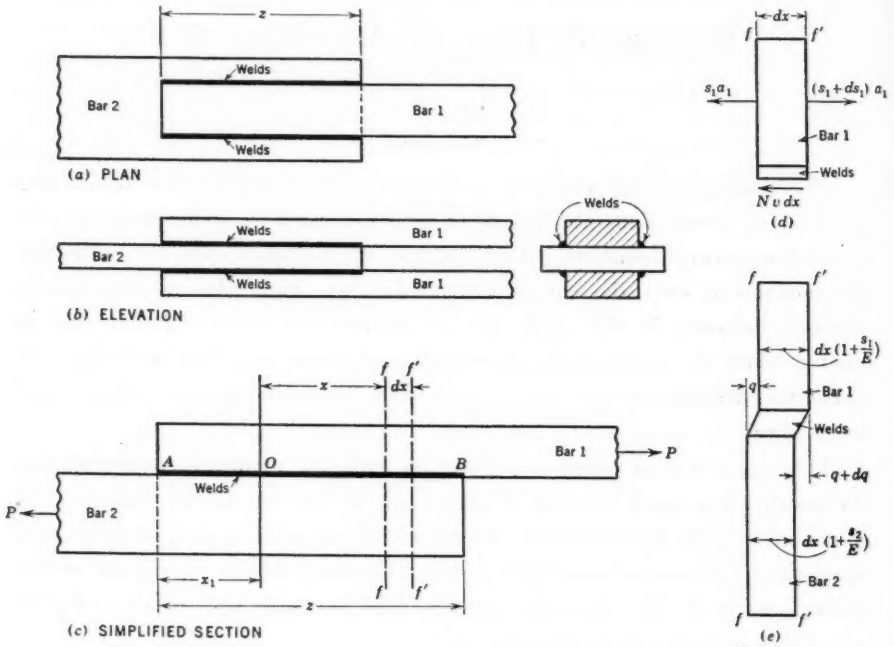

FIG. 1.

be determined experimentally. It will be measured in the same units as the modulus of elasticity, or unit stress.

The stress in Bar 1 is zero at A and increases to a maximum at B. Similarly, the stress in Bar 2 is zero at B and maximum at A. At some intermediate point, O, the unit stresses in the two bars will be equal and the bars will have equal unit deformations. Proceeding from O toward A the stress in Bar 1, and, consequently, the deformation, becomes constantly less, while in Bar 2 they grow constantly greater. It follows that the displacement between the bars becomes greater as the distance from O increases. Since this displacement is the shearing deformation in the welds, the shear in the welds increases from O to A. Similarly, the shear will increase from O to B. It is evident, therefore, that the minimum shear will occur at O, where the unit stresses in the two bars are equal.

Taking this point, O, as the origin of co-ordinates, let,

a_1 = cross-sectional area of Bar 1.
a_2 = cross-sectional area of Bar 2.
s_1 = unit stress in Bar 1 at any Section f-f.
s_2 = unit stress in Bar 2 at Section f-f.
E = modulus of elasticity in tension and compression.
N = the number of welds in the joint.
z = the length of the joint.
x_1 = distance of O from the spliced end of Bar 1.
x = abscissa of Section f-f with reference to O.
P = total stress (taken as tension) transmitted by the joint.

Consider now that part of the spliced member on one side of Section f-f. Since the sum of all horizontal forces must be zero, $s_1 a_1 + s_2 a_2 = P$; whence,

$$s_2 = \frac{P - s_1 a_1}{a_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Take a second section, f'-f', at a distance, dx, from f-f, and consider that part of Bar 1 which lies between the two sections (Fig. 1(d)). This piece is subject to a tension, $s_1 a_1$, at Section f-f, a tension, $(s_1 + ds_1) a_1$, at Section f'-f', and a shear, $N v dx$, along the welds. From the equilibrium of horizontal forces: $s_1 a_1 + N v dx = (s_1 + ds_1) a_1$, from which,

$$v = \frac{a_1}{N} \frac{ds_1}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Substituting this value for v in Equation (1), the shearing deformation is found to be,

$$q = \frac{v}{D} = \frac{a_1}{N D} \frac{ds_1}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Consider now the deformations of those parts of each bar and the welds that lie between Sections f-f and f'-f'. Under the action of the tensile stress the length, dx, will increase to $dx \left(1 + \frac{s_1}{E}\right)$ for Bar 1, and to $dx \left(1 + \frac{s_2}{E}\right)$ for Bar 2. The increment of the shearing deformation in the distance, dx, will be dq, and the deformation, q, at Section f-f will change to $q + dq$ at Section f'-f'. These relations are shown in Fig. 1(e), from which it is evident that $q + dx \left(1 + \frac{s_1}{E}\right) = q + dq + dx \left(1 + \frac{s_2}{E}\right)$; whence,

$$\frac{dq}{dx} = \frac{s_1 - s_2}{E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

By differentiating Equation (4),

$$\frac{dq}{dx} = \frac{a_1}{N D} \frac{d^2 s_1}{dx^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Equating these values of $\frac{dq}{dx}$ and substituting for s_2 its value from Equation (2),

$$\frac{d^2 s_1}{dx^2} = \left(\frac{a_1 + a_2}{a_1 a_2}\right) \frac{ND}{E} s_1 - \frac{P}{a_1 a_2} \frac{ND}{E} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

The integral of this is,

$$s_1 = K_1 \sinh \frac{x}{b} + K_2 \cosh \frac{x}{b} + \frac{P}{a_1 + a_2} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

in which,

$$b = \sqrt{\frac{a_1 a_2 E}{(a_1 + a_2) ND}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

In Equation (8), K_1 and K_2 are two constants of integration, the values of which are found as follows: From Equations (3) and (8):

$$v = \frac{a_1}{N} \frac{ds_1}{dx} = \frac{a_1 K_1}{Nb} \cosh \frac{x}{b} + \frac{a_1 K_2}{Nb} \sinh \frac{x}{b} \quad \ldots\ldots\ldots\ldots (10)$$

Since the origin, O, is at the point of minimum shear, $\frac{dv}{dx} = 0$ when $x = 0$; whence, $\frac{a_1 K_1}{N b^2} \sinh 0 + \frac{a_1 K_2}{N b^2} \cosh 0 = 0$. As $\sinh 0 = 0$ and $\cosh 0 = 1$, this reduces to $\frac{a_1 K_2}{N b^2} = 0$, or $K_2 = 0$. Equation (8) then becomes,

$$s_1 = K_1 \sinh \frac{x}{b} + \frac{P}{a_1 + a_2} \quad \ldots\ldots\ldots\ldots\ldots (11$$

At A, where $x = -x_1$, all the stress is carried by Bar 2, and $s_1 = 0$; hence, $0 = K_1 \sinh \frac{-x_1}{b} + \frac{P}{a_1 + a_2}$, or,

$$K_1 = \frac{P}{(a_1 + a_2) \sinh \frac{x_1}{b}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

At B, $x = z - x_1$, and all the stress is carried by Bar 1. Hence $s_1 = \frac{P}{a_1}$, and from Equations (11) and (12),

$$\frac{P}{a_1} = \frac{P}{(a_1 + a_2)} \frac{\sinh \frac{z - x_1}{b}}{\sinh \frac{x_1}{b}} + \frac{P}{a_1 + a_2} \quad \ldots\ldots\ldots\ldots (13)$$

Expanding $\sinh \frac{z - x_1}{b} \left[= \sinh \left(\frac{z}{b} - \frac{x_1}{b}\right)\right]$, multiplying by $\frac{(a_1 + a_2)}{P}$, and reducing,

$$\tanh \frac{x_1}{b} = \frac{a_1 \sinh \frac{z}{b}}{a_2 + a_1 \cosh \frac{z}{b}} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

which serves to determine x_1 and to locate the origin, O.

From Equations (10) and (12), and from the condition that $\frac{a_1 K_1}{N b^2} = 0$,

$$v = \frac{P a_1}{N (a_1 + a_2) b} \frac{\cosh \frac{x}{b}}{\sinh \frac{x_1}{b}}$$

Multiplying the numerator and the denominator by b, and substituting for b^2 its value from Equation (9),

$$v = \frac{P b}{a_2} \frac{D}{E} \frac{\cosh \frac{x}{b}}{\sinh \frac{x_1}{b}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The minimum shear occurs where $x = 0$, and has the value,

$$v_{\text{min.}} = \frac{P b D}{a_2 E \sinh \frac{x_1}{b}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The shear will be maximum at the ends of the joint, that is, where $x = -x_1$ and where $x = z - x_1$. When $x = -x_1$, Equation (15) becomes $v_1 = \frac{P b D}{a_2 E \tanh \frac{x_1}{b}}$, in which, v_1 is the shear at A. Giving $\tanh \frac{x_1}{b}$ its value from Equation (14),

$$v_1 = \frac{P b D}{a_2 E} \left[\frac{a_2 + a_1 \cosh \frac{z}{b}}{a_1 \sinh \frac{z}{b}} \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Similarly, letting $x = z - x_1$, and letting v_2 denote the shear at B,

$$v_2 = \frac{P b D}{a_1 E} \left[\frac{a_1 + a_2 \cosh \frac{z}{b}}{a_2 \sinh \frac{z}{b}} \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

From Equations (17) and (18), the maximum shears can be found without finding the value of x_1.

The quantities, $\frac{a_2 + a_1 \cosh \frac{z}{b}}{a_1 \sinh \frac{z}{b}}$ and $\frac{a_1 + a_2 \cosh \frac{z}{b}}{a_2 \sinh \frac{z}{b}}$, in Equations (17) and (18) are always greater than unity, and approach unity as a limit as z increases. Therefore, it is impossible by increasing z to decrease the end shears below the following limiting values:

$$v_1 = \frac{Pb}{a_2}\frac{D}{E} \quad \text{.....(19)}$$

and,

$$v_2 = \frac{Pb}{a_1}\frac{D}{E} \quad \text{.....(20)}$$

In ordinary cases the limit is approached rapidly; for example, if a_1 and $a_2 = 4$ and $\frac{z}{b} = 12$, $v_1 = 1.0000123$ times the limit.

In making tests to determine the detrusion ratio, D, it is desirable to measure the shearing deformation at the points where the actual value of the shear is equal to the average value over the entire length of the joint. Let x_a be the abscissa of such a point. The average value of the shear is $\frac{P}{Nz}$. Then, from Equation (15),

$$\frac{P}{Nz} = \frac{Pb}{a_2}\frac{D}{E}\frac{\cosh\frac{x_a}{b}}{\sinh\frac{x_1}{b}} \quad \text{.....(21)}$$

from which,

$$\cosh\frac{x_a}{b} = \frac{E}{D}\frac{a_2}{Nbz}\sinh\frac{x_1}{b} = \frac{b(a_1+a_2)}{a_1 z}\sinh\frac{x_1}{b} \quad \text{.....(22)}$$

Equation (22) gives two values of x_a which are numerically equal but of opposite sign. The direct stress at any point in Bar 1 is given by Equations (11) and (12), whence,

$$s_1 = \frac{P}{a_1 + a_2}\left[\frac{\sinh\frac{x}{b}}{\sinh\frac{x_1}{b}} + 1\right] \quad \text{.....(23)}$$

Substituting this value of s_1 in Equation (2) and reducing, the stress in Bar 2 is found to be,

$$s_2 = \frac{P}{a_1 + a_2}\left[1 - \frac{a_1\sinh\frac{x}{b}}{a_2\sinh\frac{x_1}{b}}\right] \quad \text{.....(24)}$$

In using these equations it is to be noted that x is negative when measured to the left of the origin.

Examples

Example 1.—Cross-Sectional Area of Bar 2 Twice That of Bar 1.—Let $a_1 = 4$ sq. in.; $a_2 = 8$ sq. in.; $P = 64$ kips; $N = 4$; $z = 6$ in.; $b = 1.153$; $\frac{z}{b} = 5.21$; $E = 30\,000$ kips per sq. in.; and $D = 15\,000$ kips per sq. in. Fig. 2

shows the results of applying the foregoing equations to this case. By Equations (19) and (20) the limiting values of v_1 and v_2 are found to be 4.61 and 9.22 kips per lin. in., respectively, which are only slightly less than the respective values of 4.73 and 9.30 kips given by Equations (17) and (18).

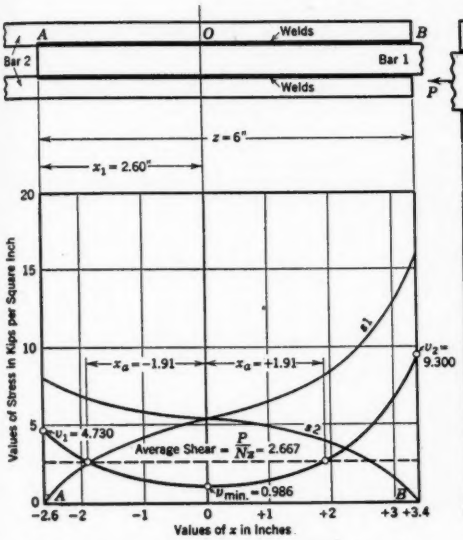

FIG. 2.—CROSS-SECTIONAL AREA OF BAR 2 TWICE THAT OF BAR 1.

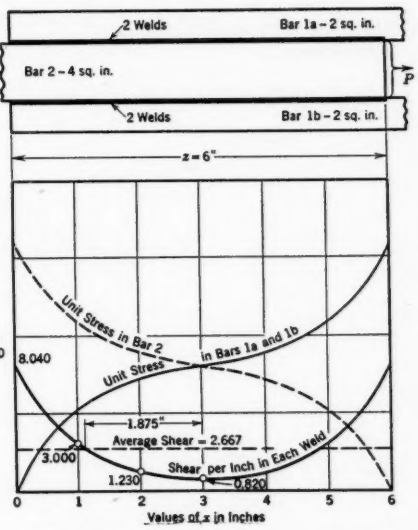

FIG. 3.—CROSS-SECTIONAL AREA OF CONNECTING BARS EQUAL.

Example 2.—Cross-Sectional Areas of Connecting Bars Equal.—Let $a_1 = a_2 = 4$ sq. in.; $z = 6$ in.; $N = 4$; $P = 64$ kips; $b = 1$; $\frac{z}{b} = 6$; $E = 30\,000$ kips per sq. in.; and $D = 15\,000$ kips per sq. in. Fig. 3 shows that, for this case, the end shear is practically equal to the limit of 8.0 kips derived by Equation (19).

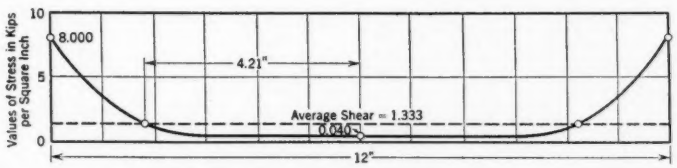

FIG. 4.—EFFECT OF INCREASING LENGTH OF WELD.

Example 3.—Effect of Increasing Length of Weld.—Let the data be the same as given in Example 2, except that $z = 12$ in. and $\frac{z}{b} = 12$. The result is shown by Fig. 4. The difference between v_1 and its limit has become negligible, and it is to be noted that the shear, for a considerable distance on each side of the center, is practically zero. This indicates (1) that the shorter the joint the more nearly uniform will be the distribution of the

shear; and (2) that beyond a certain increase in the length of weld no appreciable reduction in the maximum shear will result from further increases in length.

ACKNOWLEDGMENTS

The writer wishes to acknowledge his indebtedness to O. E. Hovey, M. Am. Soc. C. E., and F. T. Llewellyn, M. Am. Soc. C. E., for many valuable suggestions.

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PAPERS

WORK OF RIVETS IN RIVETED JOINTS

By A. Hrennikoff,[1] Esq.

Synopsis

An investigation of how a force acting on a joint, made up of plates, divides itself among the rivets, is the subject of this paper. Part I is a qualitative study, with some observations on conventional design methods. This is followed, in Part II, by the derivation of formulas for the forces developed by the rivets in a few simple types of joints, based on the assumption that a rivet develops a force proportional to deformation. In Part III the formulas in Part II are applied to the determination of the numerical values of the forces of the rivets, based on the value of the coefficients found analytically.

PART I.—Conventional Design and Actual Work of Riveted Joints

Despite their widespread use in engineering, riveted joints and the actual work in such joints when stressed, are not well understood. A rigid theoretical investigation of this subject is difficult and, in many respects, indefinite. It requires certain assumptions to start with, which may be not quite true for all cases. For example, conditions under which a rivet has been driven would affect its work considerably. A badly driven field rivet would act quite differently from a good shop rivet.

One difficulty is that inelastic deformations begin to take place a considerable time before the limit of safety is reached. Furthermore, formulas that could be developed along more or less rational lines would be difficult to use in actual practice, except in plate work (boilers, tanks, etc.). Nevertheless, the writer attempts to present herein an analysis which, while admittedly not perfect, may raise some questions of interest for discussion.

"Standard practice" has developed several arbitrary methods of design that remain in vogue mostly because they are convenient. Consider the simple case of a lap joint (Fig. 1) with two plates in tension, connected by several rows of rivets equally spaced. Imagine the joint to be divided longitudinally into equal strips with one row of rivets in each, and consider one strip as representative of the work done by the entire joint.

Note.—Discussion on this paper will be closed in **February, 1933**, *Proceedings*.

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Let w = the width of the strip; t, the thickness of each plate; d, the diameter of the rivet; p, the pitch of rivets; n, the number of rivets in each longitudinal strip; and F, the total pull on one strip.

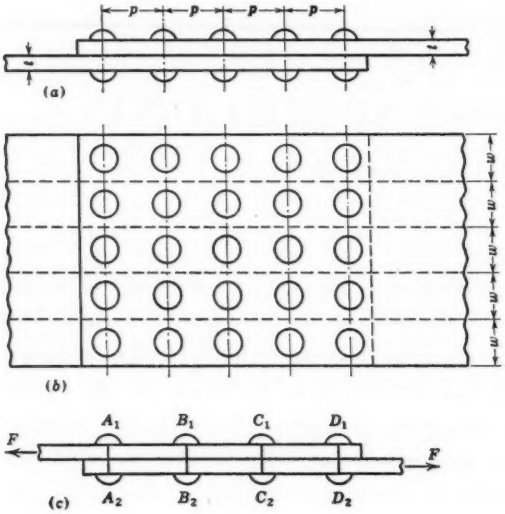

FIG. 1.

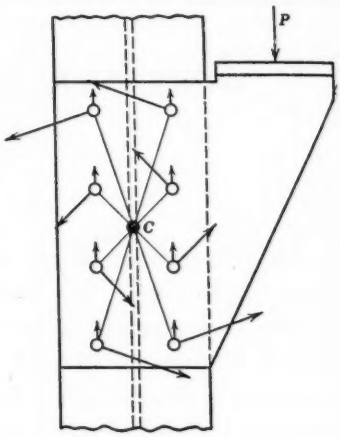

FIG. 2.

It is customary to assume that the force, F, acting on a joint, is divided equally among the rivets, so that each rivet is stressed by the force, $\frac{F}{n}$, in shear and in crushing. The unit stresses are expressed by the following equations:

For shear,

$$s_s = \frac{F}{n\,\frac{\pi d^2}{4}} = \frac{4F}{\pi d^2 n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

For crushing,

$$s_c = \frac{F}{t d n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Assuming certain safe unit stresses in shear and crushing, s_s and s_c, the number of rivets required to transmit the force, F, safely must be not less than that computed by either of the following equations:

$$n_s = \frac{4F}{\pi s_s d^2} = k_s \frac{F}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and,

$$n_c = \frac{F}{s_c t d} = k_c \frac{F}{t d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which, k_s and k_c, depending on s_s and s_c, are established by experiment.

It must be pointed out that a rivet cannot work in shear and bearing without deforming; therefore, the standard accepted theory is based on the assumption that a rivet is deformed even when it is under a small stress. Although this may be true in the case of imperfect rivets, a perfect one does not deform until the friction force between the plates, caused by tension in the shank, is exceeded.

Sometimes engineers state that a rivet is good only as long as it does not deform at all and that it should be designed on the basis of friction only. A safe force for one rivet on this basis would be also proportional to the cross-sectional area of the rivet, thus:

$$F_1 = cd^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which, c is a coefficient, depending on the coefficient of friction between the plates, temperature, and other conditions of driving the rivet, Young's modulus, and the coefficient of expansion of the metal. For the total number of rivets in a row this would give an expression similar to Equation (3) in the design for shear, namely,

$$n_f = k_f \frac{F}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which, k_f again would be established by experiment.

It must not be concluded, however, that design for shear is the same as that for friction. This simple method of designing on the basis of friction seems to be quite correct in the case of only one rivet in a longitudinal row, or two rivets in case of plates of equal thicknesses. For more than two rivets in a row the method breaks down, as may be seen if deformation of the plates is taken into account.

Let Fig. 1(c) represent a lap joint of the type shown in Fig. 1(a) with four rivets in a row. If the rivets are working in friction only the plates must not slip with respect to each other under the action of Force F; that is, the lengths, $A_1 B_1$, of the upper plate, and $A_2 B_2$, of the lower plate, which are equal before the force begins to act, must remain so during the action of force. In other words, the elongation of the section, $A_1 B_1$, in the upper plate under the force, F, must equal that of the corresponding section, $A_2 B_2$, in the lower plate. That means that the tensile forces in each plate must be equal (the plates being of the same thickness, width, and metal), which can happen only if Rivet A has transmitted to the lower plate one-half the force, F, that made the tensile forces equal in both plates in the section, A-B.

If a larger or a smaller force is transmitted by the rivet, A, the plates do not stretch equally, they slip, and at least one of the rivets deforms. Similarly, if the rivet, C, does not deform, the parts, B-C, of the upper and lower plates must stretch equally, which is possible only if the rivet, B, does no work. The same reasoning would show that Rivet C does no work either and that Rivet D transmits the other half of the force, F; that is, if the rivets work in friction only, it is really the outer rivets, one on each side of the joint, that actually work, the intermediate rivets performing no work. Hence, it follows that a method of design based on allowing a uniform value for each rivet, working in friction, is wrong.

What probably happens in the row of rivets, when the strip (Fig. 1), is subjected to a gradually increasing force, F, is that when F is small, only the outer rivets, A and D, do any work, each transmitting by friction a force, $\frac{F}{2}$. As soon as the increasing tensile force, F, equals the maximum friction force that can be developed by two rivets, the outer rivets begin to deform as the plates slip at the points, A and D.

Then the length, $A_1 B_1$, is greater than $A_2 B_2$, since $A_1 B_1$ is elongated by the greater force. This brings the rivet, B, into play in such a way that it takes up the remainder left by the rivet, A; that is $X_B = \frac{F}{2} - X_A$, in which, X_A and X_B are the forces developed by the rivets, A and B.

Force X_B is developed entirely by friction, and X_A is the sum of the forces developed by friction and shear, and crushing ($X_A = X_{Af} + X_{As}$). The force, X_{Af}, developed by friction at Rivet A probably remains more or less constant, as the tensile force continues to increase, and it is equal or (more likely) somewhat less than the maximum friction force that a rivet can develop before it begins to deform.

It is reasonable to assume that the other part, X_{As}, of the total force, X_A, developed by the rivet (that due to shear), is proportional to the slip of plates at Point A.

In the foregoing example, Rivet C works the same as Rivet B, and Rivet D works the same as Rivet A. As the tensile force, F, increases still more both X_A and X_B increase; at a certain time, the rivet, B, cannot develop its share by friction alone and it also begins to deform. The force, X_B, taken by Rivet B after that is the sum of the forces developed by friction and shear, and crushing ($X_B = X_{Bf} + X_{Bs}$). The force developed by friction, X_{Bf}, is the same as that of Rivet A. When X_B increases, the friction force, X_{Bf}, remains constant and the force developed by shear and crushing, X_{Bs}, increases in proportion to the deformation.

If the friction force, X_{Af}, and the coefficient of proportionality between the shearing force developed by a rivet and its deformation are determined by experiment, a reasonably rigid theory of a riveted joint may be deduced by means of a mathematical analysis.

PART II.—Formulas for the Forces Acting on Rivets

Following the logic of Part I, it is evident that the value of X_{Af} may vary widely for the same size of rivet, depending on how well the rivet has been driven. For a bad rivet it may approach zero. In that case, the rivet works only in shear and crushing and develops a force proportional to deformation. Formulas developed under this assumption will evidently be on the safe side. The assumption is in agreement with general practice in the design of riveted joints that transmit bending moments, as in the case of a joint in the web of a plate girder, or a bracket connection to a column (Fig. 2).

The usual procedure in these cases is to find the center of gravity of the rivets (Point C, Fig. 2) and to allow for each rivet a force, proportional to

the distance from the center of gravity, and at right angles to the line from the center of gravity to the rivet. Equating the sum of the moments of all the rivets to the external moment, the force due to bending can be found for each rivet.

This procedure is really based on two assumptions: First, that the ends of the connected parts, considering bending moment only, turn with respect to each other about the center of the rivets, due to which fact the rivets become deformed proportionally to their distances from the center of gravity of rivets; and, second, that the forces of the rivets are proportional to their deformations.

This second assumption will be adopted in the following examples for the case of a lap joint (or butt joint) of plates in direct tension or compression.

Examples

Example 1.—Lap Joint with Three Rivets.—Let Fig. 3 represent a longitudinal strip of a lap joint similar to Fig. 1(*c*), held by three rivets, *A*, *B*, and *C*. It is evident from symmetry that Rivet *A* develops the same force, *X*, as Rivet *C*; then the middle rivet, *B*, takes the remainder, or $F - 2X_1$.

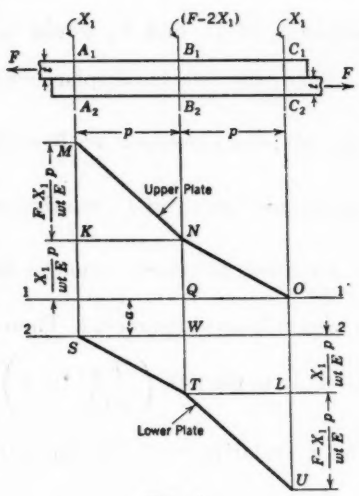

FIG. 3.

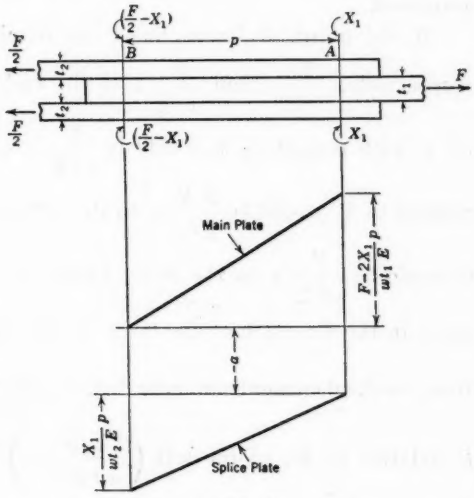

FIG. 4.

In the upper plate a tensile force, $F - X_1$, acts between the rivets, *A* and *B*, and a force, X_1, acts between the rivets, *B* and *C*. In the lower plate a tensile force, X_1, acts between A_2 and B_2 and the force, $F - X_1$, acts between B_2 and C_2.

Under these tensile forces the plates elongate. The portions, $A_1 B_1$, and $B_2 C_2$, each stretch an amount equal to $\frac{F - X_1}{w t E} p$, and the portions, $B_1 C_1$ and $A_2 B_2$, stretch an amount equal to $\frac{X_1}{w t E} p$; or, in other words, the points,

A_1 and B_1, move leftward from Point C_1, and the points, B_2 and C_2, move rightward relative to Point A_2.

Curves of displacements of different points on the plates with respect to Point C_1 on the upper plate and Point A_2 on the lower plate, are shown in Fig. 3. The ordinates represent the relative movements of points on the plates having corresponding abscissas, referred to C_1 or A_2; thus, $N\,Q = W\,T = \frac{X_1}{w\,t\,E}\,p$; and, $M\,K = L\,U = \frac{F - X_1}{w\,t\,E}\,p$. Having represented on Fig. 3 the relative displacements of points on the same plate, it is required to find how much Point A_1 will move with respect to Point A_2, B_1 with respect to B_2, and C_1 with respect to C_2.

Imagine that one point of the joint, C_1, is fixed in space. This is quite legitimate since it is the relative displacements of the points of the joint that matter. Then, Points A_1 and B_1 move, as has been seen, to the left from their original positions the distances, $\frac{F}{w\,t\,E}\,p$ and $\frac{X_1}{w\,t\,E}\,p$, respectively. Points A_2, B_2, and C_2 also move in some manner, a distance as yet undetermined.

If the point, A_2, were fixed, the displacements of B_2 and C_2 would be, respectively, $\frac{X_1}{w\,t\,E}\,p$ and $\frac{F}{w\,t\,E}\,p$ to the right. Then, the relative displacement of A_1 with respect to A_2 would be $\frac{F}{w\,t\,E}\,p$ to the left; the movement of B_1 with respect to B_2 would be $\frac{2\,X_1}{w\,t\,E}\,p$ to the left; and the movement of C_1 relative to C_2 would be $\frac{F}{w\,t\,E}\,p$ to the left. Point A_2 is not fixed, however; it moves in space in the direction of the force (to the left) an unknown distance, l. Therefore, the displacements are actually: A_1 relative to A_2, to the left, $\left(\frac{F\,p}{w\,t\,E} - l\right)$; B_1 relative to B_2, to the left, $\left(\frac{2\,X_1\,p}{w\,t\,E} - l\right)$; and C_1 relative to C_2, to the left, $\left(\frac{F\,p}{w\,t\,E} - l\right)$. (Perhaps it would be better to state, "C_2 relative to C_1, to the right" since the assumption has been made that Point C_1 was fixed; however, the two statements are mathematically equivalent.)

The relative displacements, $(A_1\text{-}A_2)$, $(B_1\text{-}B_2)$, and $(C_1\text{-}C_2)$, are measures of the deformations of the rivets, and must be of such magnitudes, as to bring into play the forces distributed among them; namely, X_1 for Rivets A and C, and $(F - 2\,X_1)$ for Rivet B. Assumed proportionality between the deformation and the force of the rivet gives the necessary equations with the unknowns, l and X_1, as follows,

$$\left(\frac{F}{wtE}\,p - l\right) = k\,X_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

and,

$$\left(\frac{2\,X_1}{wtE}\,p - l\right) = k\,(F - 2\,X_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The coefficient of rivets, k, is assumed to be known.

In Fig. 3, if the distance between the lines, 1-1 and 2-2, is $-l$, the deformation of any rivet is equal to the vertical ordinate between the curves, $M\,N\,O$ and $S\,T\,U$, taken under that rivet. This holds true for the case of a lap joint or a butt joint with any number of rivets.

From Equations (7) and (8), the force acting on Rivets A and C is,

$$X_1 = \frac{wtEk + p}{3\,wtEk + 2\,p}\,F = C_1 F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and the force acting on Rivet B is,

$$X_2 = \frac{wtEk}{3\,wtEk + 2\,p}\,F = C_2 F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which, the coefficients of F, marked C_1 and C_2, represent the fractions of the total force, F, taken up by each rivet in question. When the rivets are very stiff, k is small and approaches zero as a limit; then, $X_1 = \frac{F}{2}$ and $X_2 = 0$. On the other hand, when the rivets are very weak, k is large and approaches infinity as a limit. Then, $X_1 = \frac{F}{3}$ and $X_2 = \frac{F}{3}$; that is, when the rivets are stiff and the plates weak a relatively smaller part of the total force, F, is transmitted by the middle rivet.

If similar lap joints with four, five, and six rivets, are analyzed, the following expressions are derived for the force acting on rivets:

Lap Joint with Four Rivets:

Outer rivet,

$$X_1 = \frac{wtEk + 2\,p}{4\,wtEk + 4\,p}\,F = C'_1 F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Inner rivet,

$$X_2 = \frac{wtEk}{4\,wtEk + 4\,p}\,F = C'_2 F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Lap Joint with Five Rivets:

Outer rivet,

$$X_1 = \frac{2\,p^2 + 4\,p\,wtEk + (wtEk)^2}{4\,p^2 + 10\,p\,wtEk + 5\,(wtEk)^2}\,F = C''_1 F \quad \ldots\ldots\ldots\ldots (13)$$

Second rivet,

$$X_2 = \frac{p\,wtEk + (wtEk)^2}{4\,p^2 + 10\,p\,wtEk + 5\,(wtEk)^2}\,F = C''_2 F \quad \ldots\ldots\ldots\ldots (14)$$

Middle rivet,

$$X_3 = \frac{(wtEk)^2}{4p^2 + 10pwtEk + 5(wtEk)^2} F = C''_3 F \ldots\ldots\ldots (15)$$

Lap Joint with Six Rivets:

Outer rivet,

$$X_1 = \frac{4p^2 + 6pwtEk + (wtEk)^2}{8p^2 + 16pwtEk + 6(wtEk)^2} F = C'''_1 F \ldots\ldots\ldots (16)$$

Second rivet,

$$X_2 = \frac{2pwtEk + (wtEk)^2}{8p^2 + 16pwtEk + 6(wtEk)^2} F = C'''_2 F \ldots\ldots\ldots (17)$$

Third rivet,

$$X_3 = \frac{(wtEk)^2}{8p^2 + 16pwtEk + 6(wtEk)^2} F = C'''_3 F \ldots\ldots\ldots (18)$$

A discussion similar to that given for the case of three rivets leads to the conclusion—in the cases of four, five, and six rivets—that for very stiff rivets and weak plates the inner rivets take only a small portion of the total force, but, when the plates are strong and the rivets weak, the distribution of forces between rivets in a joint approaches equality.

Example 2.—Butt Joint with Two Splice Plates and Four Rivets Symmetrical About the Butt Joint.—Assume that the splice plates in Fig. 4 are of equal thickness, t_2, and different from the thickness, t_1, of the main plate. Let Rivet A transmit to each splice plate the force X_1; and let Rivet B transmit to each splice plate the force, $X_2 = \frac{F}{2} - X_1$. The force exerted by each rivet may be expressed as follows,

$$\frac{F - 2X_1}{wt_1E} p - l = kX_1 \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

and,

$$\frac{X_1 p}{wt_2E} - l = k\left(\frac{F}{2} - X_1\right) \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

from which,

$$X_1 = \frac{2t_2p + wt_1t_2Ek}{(4t_2 + 2t_1)p + 4wt_1t_2Ek} F = C_1'''' F \ldots\ldots\ldots\ldots (21)$$

and,

$$X_2 = \frac{t_1p + wt_1t_2Ek}{(4t_2 + 2t_1)p + 4wt_1t_2Ek} F = C_2'''' F \ldots\ldots\ldots\ldots (22)$$

From these equations it follows that if $t_1 = 2t_2$, the force is divided equally among the rivets. If $t_1 \neq 2t_2$, the rivet on the side of the greater thickness works less.

Example 3.—Case of Butt Joint with Two Cover-Plates and Six Rivets, Symmetrical About the Butt Joint.—Following the same logic as in Example 2, the forces transmitted by rivets to each cover-plate are:

$$X_1 = \tfrac{1}{2}\,\frac{2\,p^2 t_2\,(t_1 + 2\,t_2) + 6\,p\,w t_1 t_2^2 E k + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p\,w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_1'''''\,F \ldots (23)$$

$$X_2 = \tfrac{1}{2}\,\frac{p\,w t_1 t_2 E k\,(t_1 + 2\,t_2) + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p\,w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_2'''''\,F \ldots (24)$$

and,

$$X_3 = \tfrac{1}{2}\,\frac{p^2 t_1\,(2\,t_2 + t_1) + 3\,p\,w t_1^2 t_2 E k + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p\,w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_3'''''\,F \ldots (25)$$

in which, X_1 X_2, and X_3 are the forces transmitted to each cover-plate by the rivets, numbered from the outside of the joint.

Comments

Formulas similar to Equations (7) to (25) could be developed for any number of rivets in a butt joint or a lap joint with equal or unequal thickness of plates and different pitch of the rivets. The principle is always the same, the equations are of the first degree, and their number equals the number of unknowns; but the results are complicated.

One point of distinction between compression joints and tension joints may be noted. In the formulas, w signifies the gross width of the strip only in the case of a compression joint, whereas, in the case of a tension joint, it must be taken as some value between the gross and net widths, depending on the pitch of the rivets, p.

For a small pitch, such as three diameters of the rivet, it is suggested that w be taken as, $w = (W - d)$, or $w = \left(W - \frac{3}{4}d\right)$, in which, W is the gross width. For a large value of p, such as $6d$, for example, it would be reasonable, perhaps, to take $w = W - \frac{d}{4}$, approximately. Possibly the formulas becomes less accurate for tension when the values of p and w are comparatively small.

The writer has assumed tacitly that cross-sections of the joint remain plane and at right angles to the acting force. This assumption is essentially correct except at the part of the plate immediately surrounding the rivet shank. This factor, however, does not seem to be of such importance as to invalidate the formulas.

PART III.—Numerical Values of the Forces of the Rivets

To state that inner and outer rivets do work unequally does not mean much unless the amount of that inequality is known; and in order to find it the numerical value of the rivet coefficient, k, must be determined by experiment. Lacking experimental data, however, an approximate mathematical derivation has been made.

The coefficient, k, as is evident from Equation (7), is the deformation of a rivet that develops a unit force.

When deformation occurs at a rivet it is due to one or all of the following three causes: (*a*) The axis of the rivet becomes curved due to shear and moment; (*b*) the surface of the rivet becomes crushed; and (*c*) the bearing surface of the rivet hole becomes crushed and distorted. These three factors combined allow the plates to slip relative to each other. As a result of a rather long mathematical analysis (which is omitted), the following expressions were obtained for the rivet coefficients in a tension lap joint composed of plates of equal thickness, t:

$$k = \frac{1}{7\ 315\ d} \qquad \text{.....(26)}$$

and,

$$k = \frac{1}{10\ 850\ d \tanh\left(2.3\ \frac{t}{d}\right)} \qquad \text{.....(27)}$$

Equations (26) and (27) were developed by considering the deformed rivet as a beam with fixed ends, loaded along its length with a continuous load, of a magnitude at each point, proportional to the amount of crushing. For comparatively long rivets, the shear deformation is of little importance, while for the short rivets, on the contrary, the bending deformation need not be taken into account.

Accordingly, Equation (27) takes no cognizance of bending, and, with $\frac{7}{8}$-in. rivets, it should be used for values of $t = \frac{1}{2}$ in., or less, while Equation (26), developed by neglecting shear, should be used with the same diameter of rivets for values greater than $t = \frac{1}{2}$ in.

For a two cover-plate butt joint with a main plate thickness of t_1 and a cover-plate thickness of t_2, the rivet coefficient, k, must be taken as the average of the two values obtained from Equations (26) or (27), using for t the values, $t = \frac{t_1}{2}$ and $t = t_2$, respectively.

It is not claimed that these formulas for k are very accurate; nor is it believed that any formula could be derived taking all contributing factors into account. It would need to be tested experimentally. The rivet hole may be irregular; the shank of the rivet may not completely fill it; and the driven rivet may leave space around it in cooling. All these factors tend to complicate the problem of determining a mathematically accurate value of the coefficient.

Even allowing for these influences, however, and not taking the friction force of the rivet into account, the coefficient, k, must vary to some extent with the force, since inelastic deformations begin very early because of the high bearing stress on the plate in the middle of the rivet. However, the writer thinks that the values of k in Equations (26) and (27) are not entirely misleading and that they are useful for demonstrating the great differences in the work done by rivets, which ordinarily are assumed to work equally. Table 1 has been prepared for this purpose by means of Equations (9) to (25), using values of k from Equations (26) and (27). In Table 1,

C is the fraction of the total force taken by each rivet, according to conventional design (reciprocal of the total number of shears in the joint) and C_1, C_2, and C_3 are fractions of the total force taken by each rivet, as given by Equations (9) to (25), inclusive. The percentage of over-stress is listed in the last column.

TABLE 1.—VALUES OF DISTRIBUTION COEFFICIENTS, C
(Diameter of rivets = $\frac{7}{8}$ in.)

Number of rivets	Width of strip, w, in inches	Pitch of rivets, p, in inches	Thickness of Plate, t, in Inches		Stress factor, k, in inches per kip	Distribution Coefficients				Percentage of over-stress
			t_1	t_2		C	C_1	C_2	C_3	
Lap Joints										
3....	3	2.5	0.5		1/8200	0.333	0.371	0.257	.....	11.4
	3	5.0	0.5		1/8200	0.333	0.395	0.210	.....	18.6
	3	2.5	1.0		1/6400	0.333	0.351	0.298	.....	5.4
	3	5.0	1.0		1/6400	0.333	0.365	0.270	.....	9.6
4....	3	2.5	0.5		1/8200	0.250	0.327	0.173	.....	30.8
	3	5.0	0.5		1/8200	0.250	0.368	0.132	.....	47.1
	3	2.5	1.0		1/6400	0.250	0.288	0.212	.....	15.2
	3	5.0	1.0		1/6400	0.250	0.316	0.184	.....	26.4
5....	3	2.5	0.5		1/8200	0.200	0.312	0.140	0.096	56.0
	3	5.0	0.5		1/8200	0.200	0.3618	0.1095	0.0574	80.9
	3	2.5	1.0		1/6400	0.200	0.2570	0.1705	0.1450	28.5
	3	5.0	1.0		1/6400	0.200	0.2955	0.1495	0.110	47.7
6....	3	2.5	0.5		1/8200	0.1667	0.305	0.128	0.067	83.0
	3	5.0	0.5		1/8200	0.1667	0.3596	0.1036	0.0368	115.6
	3	2.5	1.0		1/6400	0.1667	0.241	0.149	0.110	44.5
	3	5.0	1.0		1/6400	0.1667	0.286	0.135	0.079	71.5
Butt Joints										
2....	3	2.5	0.5	0.375	1/6180	0.250	0.268	0.232	.....	7.2
	3	5.0	0.5	0.375	1/6180	0.250	0.277	0.223	.....	10.8
	*	*	0.75	0.375	1/7150	0.250	0.250	0.250	.....	0.0
	3	2.5	0.75	0.5	1/7645	0.250	0.262	0.238	.....	4.8
	3	5.0	0.75	0.5	1/7645	0.250	0.268	0.232	.....	7.2
	*	*	1.00	0.5	1/8200	0.250	0.250	0.250	.....	0.0
3....	3	2.5	0.75	0.5	1/7645	0.1667	0.2052	0.1252	0.1696	23.1
	3	5.0	0.75	0.5	1/7645	0.1667	0.2237	0.1003	0.1760	34.2
	3	2.5	1.00	0.5	1/8200	0.1667	0.1860	0.1280	0.1860	11.6
	3	5.0	1.0	0.5	1/8200	0.1667	0.1981	0.1039	0.1981	18.8
	3	2.5	1.0	0.75	1/7200	0.1667	0.2019	0.1363	0.1618	21.1
	3	5.0	1.0	0.75	1/7200	0.1667	0.2211	0.1152	0.1637	32.6

* Any value.

The equations herein presented, with the values given in Table 1, lead to four conclusions, as follows:

1.—The standard practice of dividing the force in proportion to the shearing areas of the rivets and disregarding the deformation of the plates leads to results that are very inaccurate. In a 6-rivet lap joint the over-stress of an outer rivet designed by the conventional method may be more than 100%, and in a joint with a greater number of rivets, the over-stress seems to be still greater.

2.—Actually, the total force acting on a riveted joint is not distributed equally among the rivets; a larger proportion of the total work is done by the outer rivets.

3.—The proportion of total work in the outer rivets increases as the pitch and diameter of the rivets increase and the cross-section of the plates decreases.

4.—If there are many rivets in a longitudinal row the inner rivets are inefficient, and an increase in the number does not improve the value of the joint appreciably. For example, in Table 1, comparing the values for 5-rivet and 6-rivet joints, it is seen that when the number of rivets is increased from five to six (that is, by 20%), the force on the outer rivet decreases only by 2.3%, 0.6%, 6.6%, and 3.3%, in the four cases given.

It seems likely that in joints containing twenty or more rows of rivets, some of those in the middle perform no work and those on the outside are overstressed greatly. However, no engineering structure has ever failed because of this lack of uniform distribution, due to the ductile nature of structural steel. After the outer rivets have been stressed beyond the elastic limit, they do not break, but deform almost without increase in stress, and this brings the inner rivets into play. If the rivets were made of some brittle material, the inaccuracy of the conventional design method would soon manifest itself in the failure of the rivets.

Another factor that helps the outer rivets is friction between plates, which has been disregarded entirely in this analysis. Actually, as was seen in the beginning of this paper, the rivets do part of their work by friction which involves no deformation and retards the moment when the stress in a rivet exceeds the elastic limit.

The method of distributing a force through the rivets of a joint as presented in this paper, might prove of practical value in the design of boilers, tanks, and similar structures. The necessary coefficients could be computed quite easily for standard values of pitch, width of plate, thickness of plate, diameter of rivet, and for type of joint. Such data could then be plotted into curves for the uses of the designer. As stated previously, the rivet coefficient, k, would be determined by experiment.

Of course, the greatest disadvantage of the method is that it is less economical than the present standard method of equal distribution. Furthermore, the writer does not recommend it for general structural design of buildings, bridges, etc. In this field, serious complications arise because of the variable pitch and spacing of rivets, because of the use of structural shapes other than plates, because of the irregular shape of gusset-plates, and because of several members meeting at the same point.

However, the writer ventures to offer for discussion a few suggestions concerning the standard design of connections of structural members, in the hope of promoting conditions that will insure more uniform distribution of forces among the rivets, namely:

(1) In compression members, the number of gauge lines should be increased. This would reduce the number of rivets on each gauge line, and hence the inequality of forces taken up by the rivets. Naturally, it would be more difficult to apply this recommendation to the tension members, due to the consideration of the net section.

(2) A minimum allowable longitudinal pitch should be established, since the inequality decreases with the decrease of pitch. Although this is being

generally done in common structural practice, sometimes the pitch is lengthened due to a desire to eliminate an extra cut on the gusset-plate.

(3) The number of rivets to be placed on a gauge line should be more or less proportional to the cross-sectional areas at those gauge lines. Thus, in Fig. 5, the number of rivets on Gauge Lines 1 and 5, are just sufficient

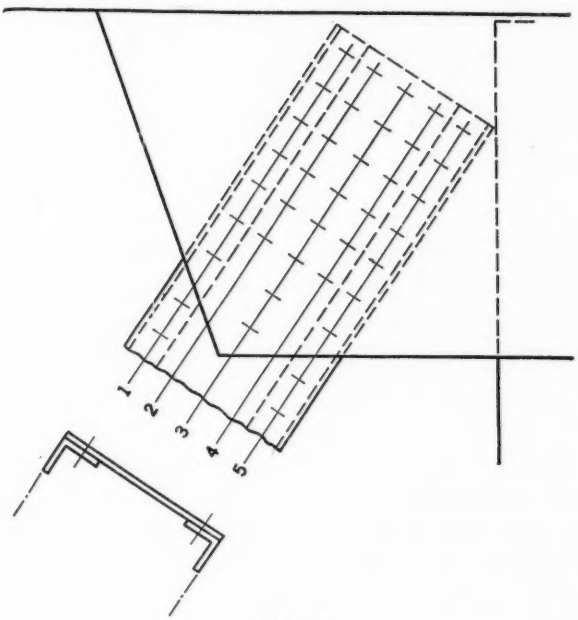

FIG. 5.

to develop the angles. These rivets are over-stressed because of the greater cross-sectional areas that are related to these gauge lines, including angles and parts of the plate.

ACKNOWLEDGMENT

While this problem has been treated in a general way previously,[2] the methods developed in this paper resulted from independent studies.

The writer wishes to acknowledge his indebtedness to A. H. Finlay, Assoc. M. Am. Soc. C. E., for invaluable guidance and criticism during the preparation of this paper.

[2] "Theory and Practice of Modern Framed Structures" by Messrs. Johnson, Bryan, and Turneaure, Vol. 3, Chapter 5; also "Structural Engineering" by the late George F. Swain, Past-Presdent, Am. Soc. C. E., p. 407.

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PAPERS

STUDY OF STILLING-BASIN DESIGN

By C. Maxwell Stanley,[1] Jun. Am. Soc. C. E.

Synopsis

This paper presents the results of a study, made by means of hydraulic models, to rationalize the design of stilling basins, one of the several devices used for the prevention of scour at the toe of a dam. The relationships between the fundamental factors controlling the action of the stilling basin are carefully compared and, finally, a method of design based upon these relationships is presented.[2]

The stilling basin may be defined broadly as a pool of water held at the toe of a dam by a weir. Five distinct types of stilling-basin action and their characteristics are classified. The experimental data from the tests are studied to determine the relationships of basin length and depth, dam height, and quantity occurring for each of the five types of action. A more complete form of the paper has been placed in Engineering Societies Library, 33 West 39th Street, New York, N. Y., for reference.

Problem of Scour

The prevention of scour at the toe of an over-flow dam is a problem of importance to the designer, and upon its successful solution depends the safety of the dam. Although the solution of the problem is more urgent in dams constructed upon foundations of sand and gravel, which are easily eroded, it must not be overlooked in dams constructed upon seemingly solid rock. Engineering literature contains too many reports of damage from erosion requiring costly repairs, if not resulting in disastrous failures of the dam itself.

A problem of such importance has naturally been studied to a considerable extent, but the theoretical foundation upon which to base an attack is obscure and little has been done to formulate the basic laws. No general mathematical formula has been developed, which will allow even an approximate solu-

Note.—Discussion of this paper will be closed in **February, 1933**, *Proceedings*.

[1] Young & Stanley, Inc., Engrs., Muscatine, Iowa.

[2] Prepared from a thesis presented to the University of Iowa in partial fulfillment of the requirements for the degree of Master of Science.

tion. As a result, solutions have been largely empirical, based on judgment and past experience. Their insufficiency has been indicated by the many cases of serious erosion, and although there have been notable examples of successful solution, the net result can scarcely be considered satisfactory.

Within recent years, many satisfactory solutions have resulted from model studies, which are to be highly commended and encouraged. The day is not far distant when experiments with models will be a necessary part of the design of every over-flow dam.

Even if a model study may be counted upon to solve a particular problem some rational method of design for protective devices would be extremely welcome. Such a method would be of great value in preliminary designs and estimates even if supplemented later by a model study.

The Stilling Basin

For the purposes of this paper, a stilling basin is defined as a pool of water retained below the toe of a dam by a weir, in order to dissipate energy contained in the water overflowing the spillway and thus to prevent serious erosion. The floor of the basin is assumed to be of concrete or other material that will prevent erosion at the toe where the sheet of water impinges. It should be pointed out, however, that natural conditions may create a pool of such dimensions that a weir is unnecessary, and the resulting actions will be similar to that described herein. A stilling basin thus defined may be deep or shallow, long or short. Naturally, the action will not be the same under these varying conditions of length and depth. The different actions fall into five types, each of which has certain characteristics differentiating it from the others, namely: I.—Roller Action; II.—Jump Action; III.—Incomplete Jump Action; IV.—Pool Action; and, V.—Spray Action.

Type I.—Roller Action.—As the swift stream of water coming down the face of the dam enters the pool of water in Fig. 1, it is deflected by the bucket and turned to a general horizontal direction. Friction on the floor and on the water above decreases the velocity and sets up an eddy or roller about a horizontal axis. This roller is full of air bubbles and slopes backward on top, slightly toward the incoming sheet of water. It absorbs and dissipates energy from the incoming sheet. The turbulence is out of the water before it reaches the weir, and the depth over the weir is just sufficient to discharge the incoming quantity. The only energy remaining is that of the static and velocity heads just above the weir, which is small compared to the incoming energy.

Type II.—Jump Action.—In the action of Type II (Fig. 2), the basin is longer than that of Type I, but in proportion to the quantity of water handled is not as deep. The general appearance is that of a hydraulic jump, as the usual indications of the jump are present. The basin is of sufficient length to allow the complete formation of this jump, with the result that all turbulence is out of the water before it reaches the weir, and it flows over smoothly. The remaining energy is that of the static and velocity heads just above the weir.

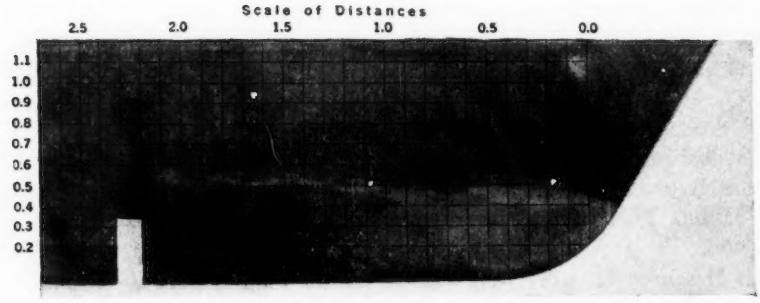


FIG. 1.—TEST 125, TYPE I: STILLING BASIN, ROLLER ACTION.

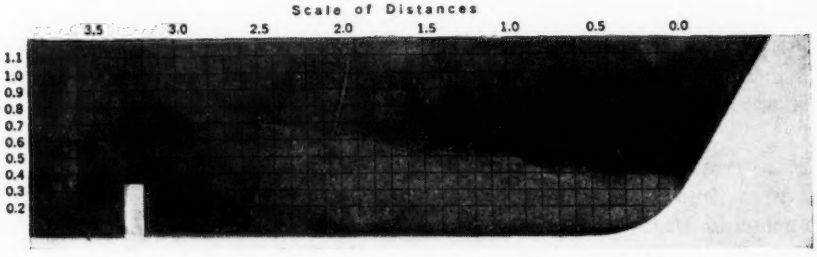


FIG. 2.—TEST 476, TYPE II: STILLING BASIN, JUMP ACTION.

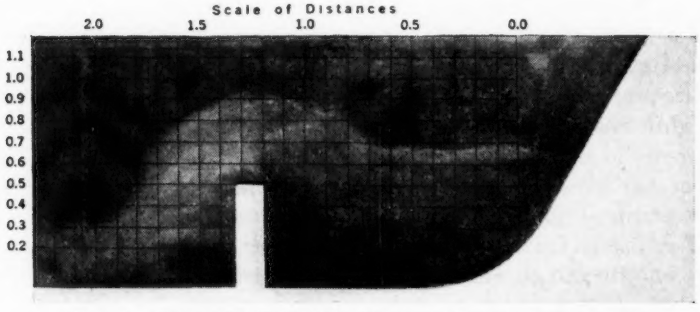


FIG. 3.—TEST 462, TYPE IV: STILLING BASIN, POOL ACTION.

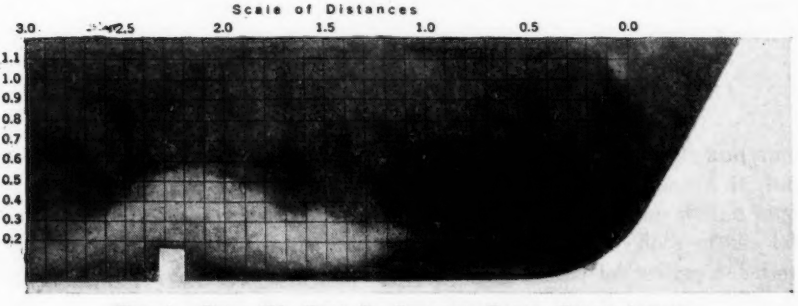


FIG. 4.—TEST 453, TYPE V: STILLING BASIN, SPRAY ACTION.

Type III.—Incomplete Jump Action.—This type of action is that occuring if the weir of Fig. 2 is moved toward the dam. The part toward the dam still gives every indication of being hydraulic-jump action, but the length is not sufficient to allow the complete formation of the jump.

The depth at the weir does not increase appreciably, but the water, instead of flowing smoothly over the weir, shoots in a more or less horizontal direction. The length clearly is too short for the proper formation of the jump. The remaining energy will be greater than that of Types I and II as the velocity over the weir is greater.

Type IV.—Pool Action.—Action of this type (Fig. 3) occurs if the weir of Fig. 1 is moved toward the dam. The basin is comparatively short and deep.

Instead of the smooth action of Type I the water shoots in and out of the basin and is thrown into the air. The pool surface is generally smooth except for a decided bump over the weir, and a slight roller is formed. The remaining energy is again greater than that of Types I and II as the water is thrown high into the air.

Type V.—Spray Action.—This type of action is illustrated in Fig. 4 and occurs if the basin is too shallow to hold the water. The length is of little importance as the sheet moves swiftly across the floor until it encounters the weir. It is then sprayed into the air. In this type, too, the remaining energy at the weir is greater than that of Types I and II.

These five types of stilling basin seem to classify the forms of action satisfactorily. The dividing lines between the types naturally are not clear cut and distinct, because one type merges into another. In Types I and II as contrasted to Types III, IV, and V, the basin is of such proportions that sufficient energy is dissipated to allow the water to flow smoothly over the weir. The energy remaining is definitely calculable, and its effects may easily be determined. Types I and II, therefore, are considered the more satisfactory kinds of action. It is the purpose of this study to define the conditions under which each of the types of action will occur and then to present a method of design by which a stilling basin may be proportioned to obtain any desired type of action.

While the ultimate test of the effectiveness of any stilling basin is its ability to prevent dangerous scouring, a clear knowledge of the factors that affect the operation of the basin and the relationships existing between these factors for satisfactory or unsatisfactory action will be helpful.

Test Methods

The experiments were performed in a timber flume, 2 ft. wide and approximately 20 ft. long, with a glass-sided flume a little more than 6 ft. long at the lower end. (See Fig. 5.) Water was supplied by a pump with a capacity of 1 000 gal. per min., from a measuring basin under the floor of the laboratory to a tank (constant level) on the hill back of the laboratory. From this tank, the water flowed through a pipe line into the flume, over the model dam, and back into the measuring basin. Cross-connections were provided

so that direct circulation could be used on the larger quantities. A Venturi meter was inserted in the pipe line and a valve was placed at the inlet to the flume. Four sets of baffles, set about 13 ft. up stream from the model dam, successfully smoothed the flow. A hook-gauge well was located 6 ft. up stream from the model.

The dam was built of wood with a cross-section as shown for Series *B* (Fig. 5). Its surface was smoothed and painted carefully with several coats of shellac. Alterations were made at the toe to obtain the cross-sections of Series *A* and *C* (Fig. 5). The weirs that formed the stilling basins were con-

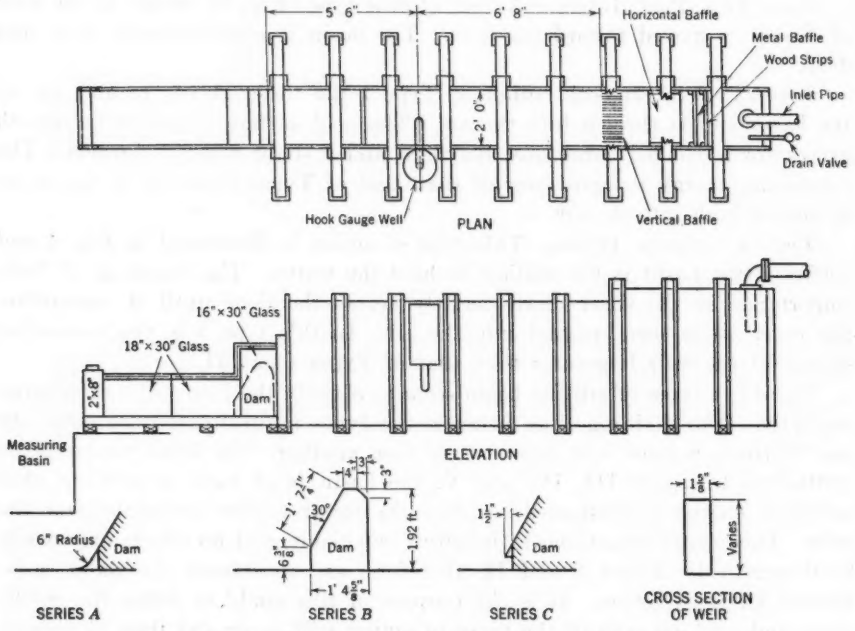


FIG. 5.—DETAILS OF TIMBER FLUME AND MODELS.

structed of 2-in. material and were fastened to the floor of the glass flume. Their rating curves, determined experimentally, are shown in Fig. 6, in which, D_w denotes depth of water at the weir (height of weir plus head on the weir) required for smooth flow over the weir, and D_j is the depth of water required in the basin for the formation of a hydraulic jump.

In running the tests a weir of given height was fastened to the floor at a given distance from the dam. The water was then circulated through the system and the quantity over the dam was adjusted by the intake valve. After conditions had become steady, readings were taken of the quantity and depth, and notes were made of the stilling-basin action. During the first runs, the Venturi meter was used to determine the quantity of water flowing over the dam and simultaneous readings were taken of the hook-gauge. As soon as a satisfactory rating curve had been built up for the dam, the Venturi meter readings were discontinued, except as an occasional check.

All readings of depth and length were taken by direct observation of the water surface through the glass by a grid painted on the glass with horizontal and vertical lines spaced 0.1 ft. apart. Notes were recorded regarding the action of the stilling basin and from these data the action was classified into the five types previously described. On many tests, sketches were made of the action in the stilling basin, and, in some cases, photographs were taken.

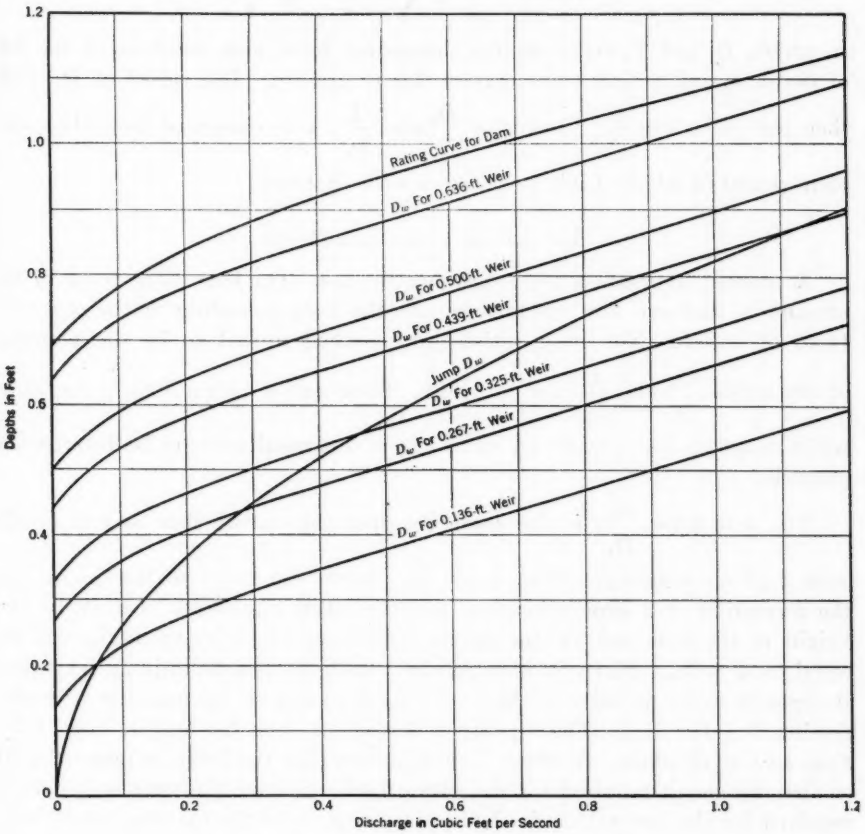


FIG. 6.—RATING CURVES FOR DAM AND WEIRS.

The tests were divided into three series (*A*, *B*, and *C*) for three variations in the form at the toe of the dam. (See Fig. 5.) The method of testing was the same in each series. The data include the height of weir, test number, length, quantity, and type of action from the observed notes. The computed values of D_j, D_w, and the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, are also shown, in which, L is the length of the basin. Table 1 illustrates the manner in which data for 350 tests were assembled and arranged for use in this study.

Rating curves were built up experimentally for weirs of different heights and from these the values of D_w were obtained. The depth of water required for the formation of a hydraulic jump for various discharges was computed (assuming no friction on the down-stream face of the dam) by the hydraulic-jump formula:

$$D_j = -\frac{D_1}{2} \pm \sqrt{\frac{2\,V_1^2\,D_1}{g} + \frac{D_1^2}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, D_1 and V_1 represent the theoretical depth and velocities at the toe of the dam, and g is the acceleration due to gravity. The values of D_j were then plotted in Fig. 6. The ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, were computed from the complete record of which Table 1 is a condensed example.

Method of Comparing Tests

A number of methods of comparing the test data have been tried in an attempt to find one that would segregate the tests according to the observed types of action. The most satisfactory method proved to be the plotting of two ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, for each test. These ratios have a definite meaning which must be kept clearly in mind if the test results are to be interpreted properly.

The first ratio, $\frac{D_w}{D_j}$, is the depth required for smooth flow over the weir with a given discharge, divided by the depth required in the basin for the formation of a hydraulic jump with the same discharge. Let W be the height of the weir and h_w, the required head on the weir; then, D_w will be equal to $W + h_w$. Since D_j is determined from the hydraulic-jump formulas, it depends on the velocity at the toe of the dam and on the quantity of water flowing over the dam. The velocity is dependent upon the height, slope, friction, etc., of the dam. A ratio of unity means that the basin is proportioned so that the depth required for the given discharge over the weir is just that required for the formation of a hydraulic jump under the existing conditions. Ratios greater or less than unity mean that the basin is deeper or shallower than required for the formation of a jump.

The other ratio, $\frac{L}{D_w}$, is the length of the basin divided by the depth required at the weir and is an index of the remainder of the proportions of the basin. When $\frac{D_w}{D_j}$ is unity, the second ratio, $\frac{L}{D_w}$, becomes $\frac{L}{D_j}$, or the length divided by the depth required for the formation of the jump.

These ratios thus tie the action of the stilling basin closely to the fundamental factors of the hydraulic jump.

TABLE 1.—TYPICAL ARRANGEMENT OF OBSERVED AND COMPUTED STILLING-BASIN DATA, SERIES *A*

Test No.	Observed			Computed				
	Length, in feet.	Discharge, in cubic feet per second	Depth, D, in feet.	Depth for hydraulic jump, D_j, in feet	Depth for weir, D_w, in feet	Ratio, $\frac{D_w}{D_j}$	Ratio, $\frac{L}{D_w}$	Action
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Weir, 0.136 foot high								
322	1.2	0.169	0.26	0.34	0.27	0.79	4.45	II
323	1.2	0.243	0.34	0.41	0.30	0.73	4.00	V
325	1.2	0.667	0.85	0.68	0.43	0.63	2.79	V
326	1.7	0.160	....	0.33	0.26	0.79	6.55	II
328	1.7	0.197	....	0.37	0.28	0.76	6.07	V
329	1.7	0.334	....	0.48	0.33	0.69	5.15	V
330	2.2	0.176	0.28	0.35	0.27	0.77	8.15	II
331	2.2	0.238	0.30	0.40	0.30	0.75	6.60	V
453	2.2	0.345	....	0.48	0.33	0.69	6.65	V
332	3.7	0.165	0.37	0.34	0.27	0.79	13.7	II
333	3.7	0.195	0.30	0.37	0.28	0.76	13.2	II
336	3.7	0.225	....	0.39	0.29	0.74	12.7	V
Weir, 0.267 foot high								
272	0.7	0.042	0.33	0.15	0.31	2.06	2.26	I
273	0.7	0.079	0.39	0.22	0.34	1.54	2.06	I
274	0.7	0.260	0.63	0.42	0.43	1.02	1.63	IV
267	1.2	0.078	0.36	0.22	0.34	1.54	3.53	I
268	1.2	0.127	0.38	0.30	0.38	1.26	3.16	I
269	1.2	0.173	0.42	0.35	0.40	1.14	3.00	IV
271	1.2	0.522	0.75	0.60	0.52	0.87	2.31	IV
262	1.7	0.117	0.38	0.28	0.37	1.32	4.60	I
263	1.7	0.206	0.43	0.38	0.41	1.08	4.15	I
264	1.7	0.308	0.47	0.46	0.45	0.98	3.78	IV
266	1.7	0.652	0.72	0.67	0.56	0.84	3.04	IV
255	2.2	0.153	0.39	0.33	0.39	1.18	5.64	I
256	2.2	0.208	0.42	0.38	0.41	1.08	5.36	II
258	2.2	0.413	0.52	0.53	0.48	0.91	4.58	II
261	2.2	0.847	0.72	0.76	0.62	0.82	3.55	III
225	2.7	0.103	0.36	0.26	0.36	1.38	7.50	I
226	2.7	0.169	0.40	0.34	0.40	1.18	6.75	I
226	2.7	0.302	0.44	0.45	0.45	1.00	6.00	II
229	2.7	0.461	0.50	0.56	0.50	0.89	5.40	II
230	2.7	0.543	0.53	0.61	0.52	0.85	5.18	III
235	2.7	1.220	0.85	0.90	0.74	0.82	3.65	III
236	3.2	0.290	0.44	0.44	0.44	1.00	7.27	II
239	3.2	0.503	0.52	0.59	0.51	0.86	6.27	II
240	3.2	0.585	0.55	0.64	0.54	0.84	5.93	III
242	3.2	0.958	0.78	0.81	0.65	0.80	4.93	III
243	3.7	0.059	0.33	0.18	0.33	1.64	11.2	I
244	3.7	0.181	0.41	0.35	0.40	1.14	9.3	II
247	3.7	0.538	0.53	0.61	0.53	0.87	7.00	II
248	3.7	0.684	0.56	0.69	0.57	0.83	6.50	V
249	3.7	0.894	0.58	0.78	0.63	0.81	5.88	V
250	4.2	0.248	0.43	0.41	0.43	1.05	9.78	II
252	4.2	0.496	0.52	0.58	0.51	0.88	8.25	II
253	4.2	0.591	0.52	0.64	0.54	0.84	7.79	V
254	4.2	0.772	0.53	0.73	0.59	0.81	7.13	V
Weir, 0.325 foot high								
143	0.7	0.056	0.40	0.18	0.38	2.11	1.84	I
142	0.7	0.118	0.47	0.28	0.42	1.50	1.67	IV
145	0.7	0.785	1.27	0.74	0.66	0.89	1.06	IV
137	1.2	0.111	0.44	0.27	0.42	1.55	2.86	I
138	1.2	0.156	0.50	0.33	0.45	1.36	2.67	IV
140	1.2	0.785	1.15	0.74	0.65	0.88	1.85	IV

Test Results, Series *A*

Each test of Series *A* has been plotted in Fig. 7, according to its value of the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, with the resulting type of action designated by the mark used in plotting the data. It is easy to draw dividing lines between the five types of action. This is proof that there are definite types of stilling-basin action and that these may be segregated according to the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$.

The solid curve in Fig. 7 marks the division between Types I and II which are above the line, and Types III, IV, and V, which are below it.

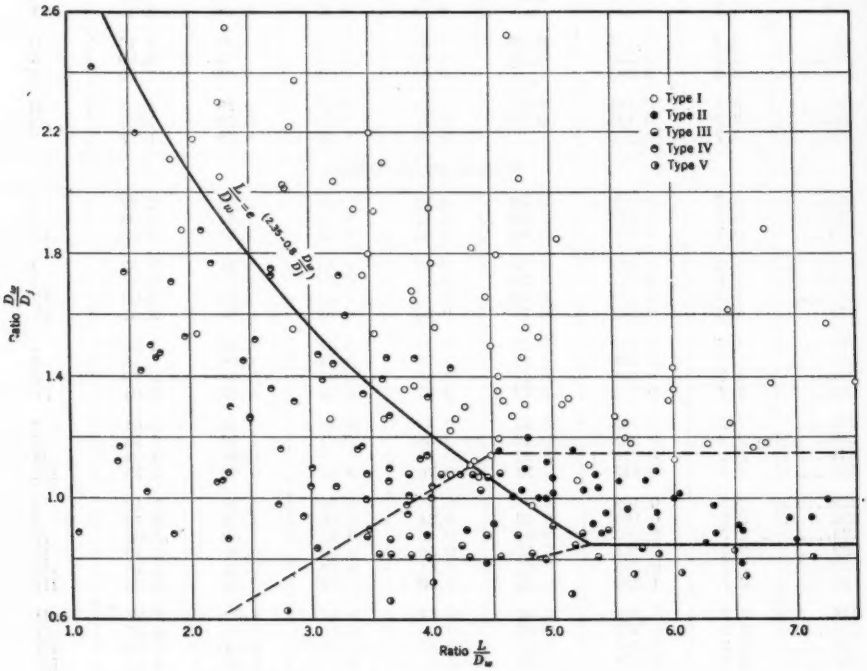
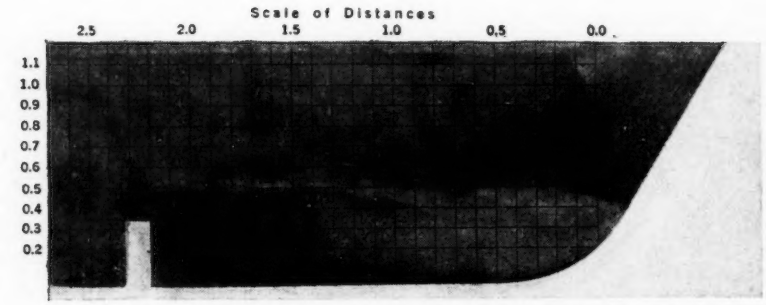


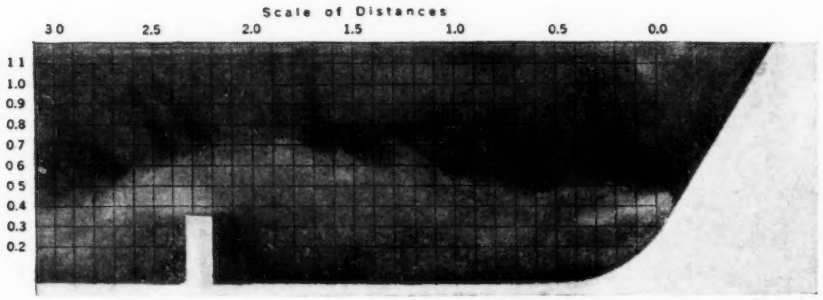
Fig. 7.—Ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, Series *A*.

Types I and II may be considered satisfactory in that stilling-basin dimensions have such proportions that the excess energy is absorbed completely from the water, allowing it to flow smoothly over the weir. In Types III, IV, and V, these requirements are not fulfilled, and the basin does not function satisfactorily.

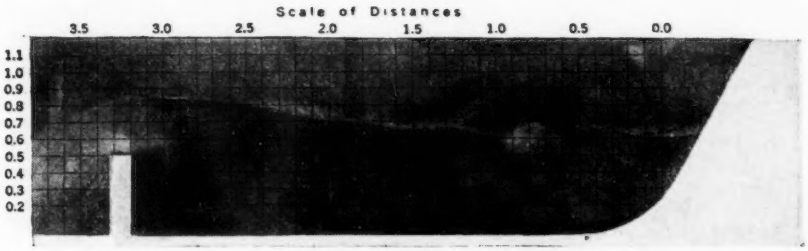
An analysis of Fig. 7 shows that Type I may be expected to occur whenever the dimensions of the bay are such that the point plots above the curve and above the line, $\frac{D_w}{D_j} = 1.15$. The action classified as Type II occurs when the



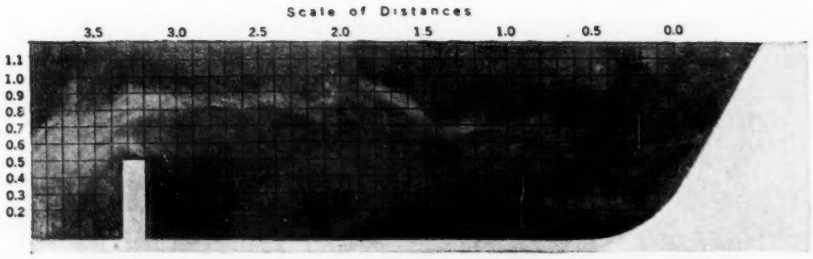
(*a*) SET 1, TEST 125, TYPE I: $L = 2.2$; $W = 0.325$; $Q = 0.193$.



(*b*) SET 1, TEST 129, TYPE IV: $L = 2.2$; $W = 0.325$; $Q = 0.720$.

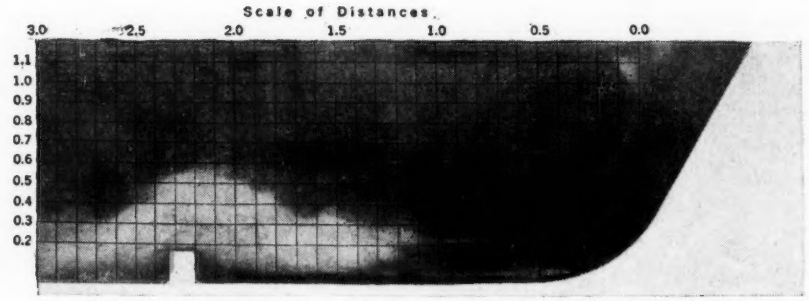


(*c*) SET 3, TEST 477, TYPE I: $L = 3.2$; $W = 0.500$; $Q = 0.610$.

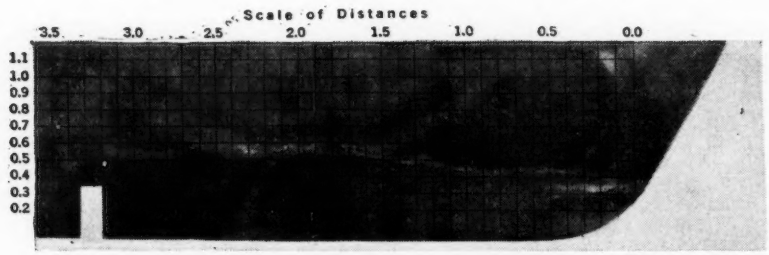


(*d*) SET 3, TEST 478, TYPE IV: $L = 3.2$; $W = 0.500$; $Q = 0.868$.

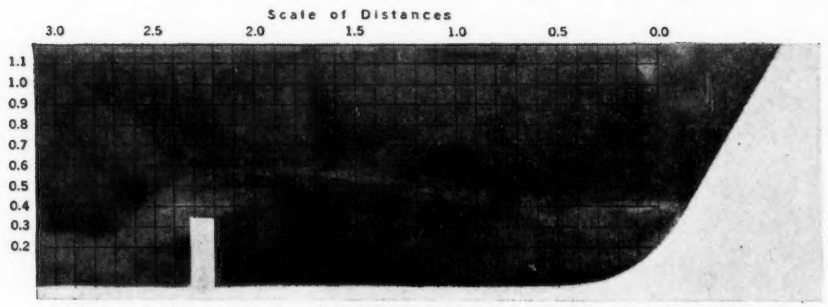
FIG. 8.—EFFECT OF INCREASING THE DISCHARGE, Q.



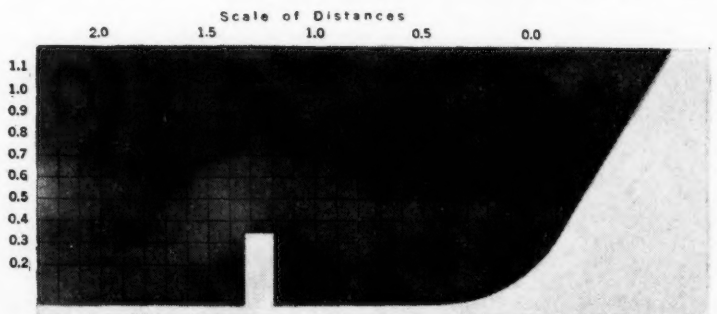
(*a*) SET 4, TYPE V: $L = 2.2$; $W = 0.136$; $Q = 0.346$; ALSO, SET 7.



(*b*) SET 5, TYPE II: $L = 3.2$; $W = 0.325$; $Q = 0.346$; ALSO, SET 2.

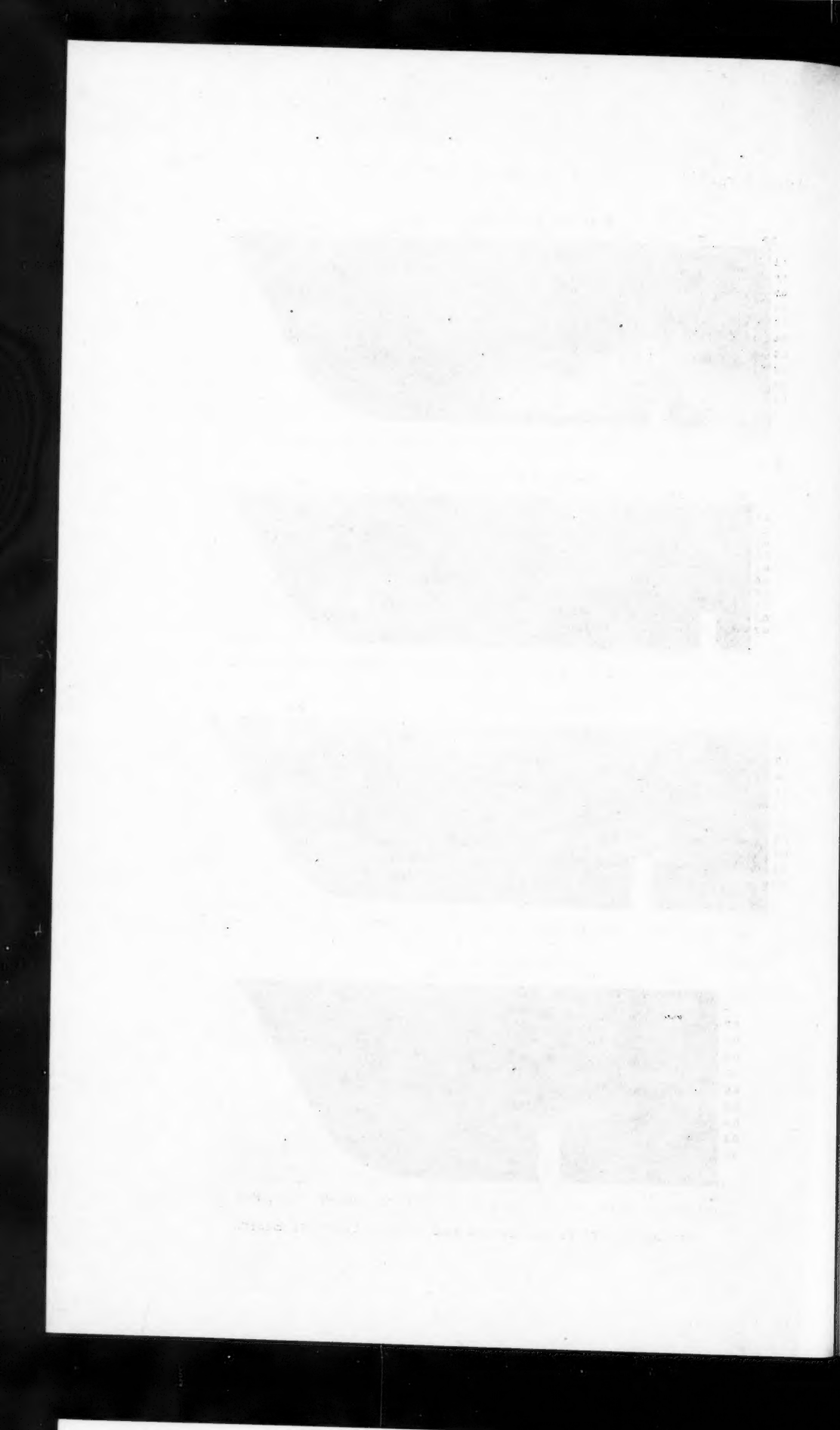


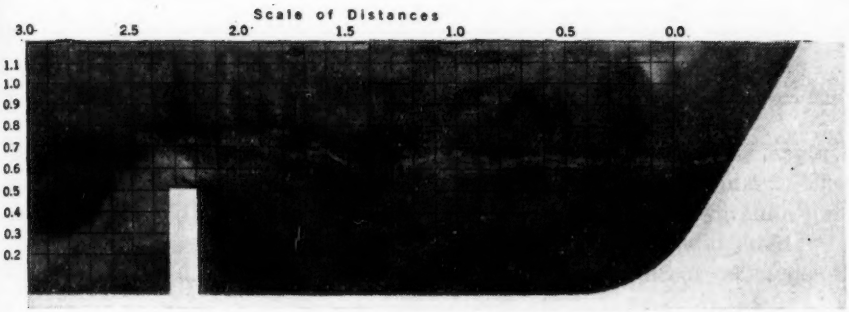
(*c*) SET 5, TYPE IV: $L = 2.2$; $W = 0.325$; $Q = 0.346$; ALSO, SETS 1 AND 7.



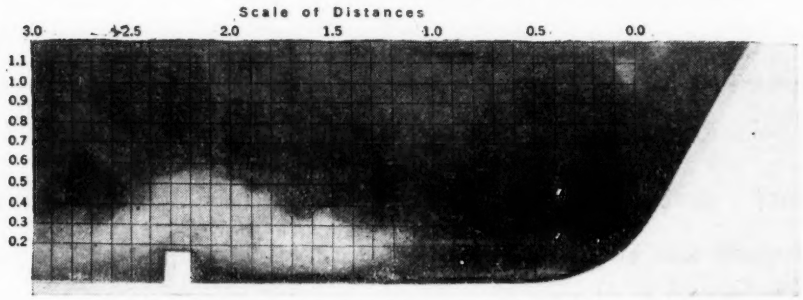
(*d*) SET 5, TYPE IV: $L = 1.2$; $W = 0.325$; $Q = 0.346$; ALSO, SET 8.

FIG. 9.—EFFECT OF DECREASING THE LENGTH, L, OF THE BASIN.

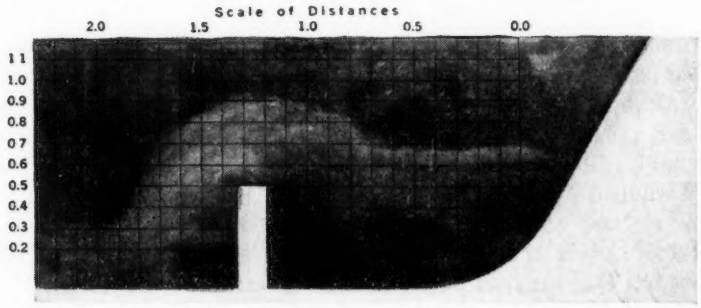




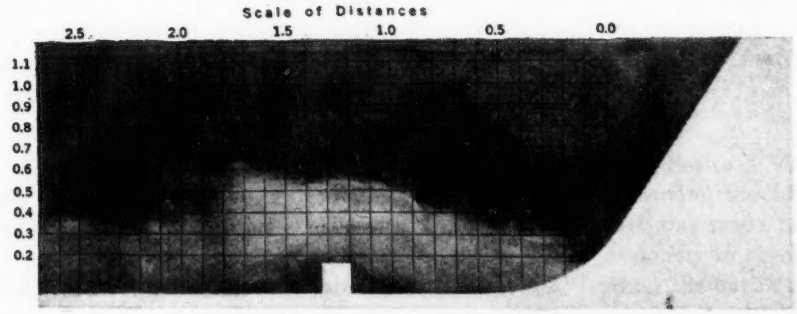
(*a*) SET 7, TYPE IV: $L = 2.2$; $W = 0.500$; $Q = 0.346$; ALSO, SET 6.



(*b*) SET 7, TYPE V: $L = 2.2$; $W = 0.136$; $Q = 0.346$; ALSO, SET 4.



(*c*) SET 8, TYPE IV: $L = 1.8$; $W = 0.500$; $Q = 0.346$; ALSO, SET 6.



(*d*) SET 8, TYPE V: $L = 1.2$; $W = 0.136$; $Q = 0.346$.

FIG. 10.— EFFECT OF DECREASING HEIGHT OF WEIR, *W*.

ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, or substantially equal to unity, and the ratio, $\frac{L}{D_w}$, exceeds about 4.5. In other words, the depth of the pool must be that required for the formation of a hydraulic jump and the length of the pool must be not less than about $4\frac{1}{2}$ times the depth of the jump. These values check with the commonly accepted requirements of a hydraulic jump, thus confirming the idea that Type II is essentially a hydraulic jump.

Types I and II, which are considered satisfactory, occur whenever the ratios plot above the curve until it reaches a value of $\frac{L}{D_w} = 5.31$. After that, the curve is a straight line, $\frac{D_w}{D_j} = 0.85$. Whenever the ratios plot below this curve, action will be unsatisfactory and will be of the classes characterized as Types III, IV, or V. Type III occurs when the ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, and $\frac{L}{D_w}$ is slightly less than that required for a jump. Type IV action occurs whenever the length or the depth is below that required for Type I. Type V action occurs whenever the ratio, $\frac{D_w}{D_j}$, is less than about 0.85 and occurs regardless of the length of the pool.

Figs. 8, 9, and 10 have been prepared to show the effect upon the basin action of changes in the fundamental variables. It is to be noted particularly that the tests are grouped in sets and that Figs. 9(*a*), 9(*b*), and 9(*c*) are parts of two sets; thus, Set 1 comprises Figs. 8(*a*), 9(*c*) and 8(*b*); Set 2 is represented by Fig. 9(*b*), as an example; Set 3 is represented by Figs. 8(*c*) and 8(*d*); Set 4 is represented by Figs 9(*a*) and 10(*b*); Set 5 includes Figs. 9(*b*), 9(*c*), and 9(*d*); Set 6 comprises Figs. 10(*c*) and 10(*a*); Set 7 is represented by Figs. 9(*c*), 10(*a*), and 10(*b*); and Set 8 is shown in Figs. 10(*c*), 9(*d*), and 10(*d*). In each set only one variable has been changed and the others have been held constant. In Fig. 11, the ratios applying to the illustrated tests are plotted and are studied in relation to the dividing line between satisfactory and unsatisfactory action in Series *A*. A study of the views and curves clearly defines the effect of changes in length, depth, height of weir, and discharge.

Test Results, Series *B*

The tests of Series *B* were performed similar to those of Series *A*, except that the 6-in. radius at the toe of the dam was removed, leaving the dam in its original cross-section as shown on Fig. 5. No attempt was made to run as complete a set of tests with Series *B*, as it was desired merely to check the curve of Series *A*. Nevertheless, the tests were run covering the entire range used in Series *A*. Fig. 12 shows the tests of Series *B* plotted with ratios

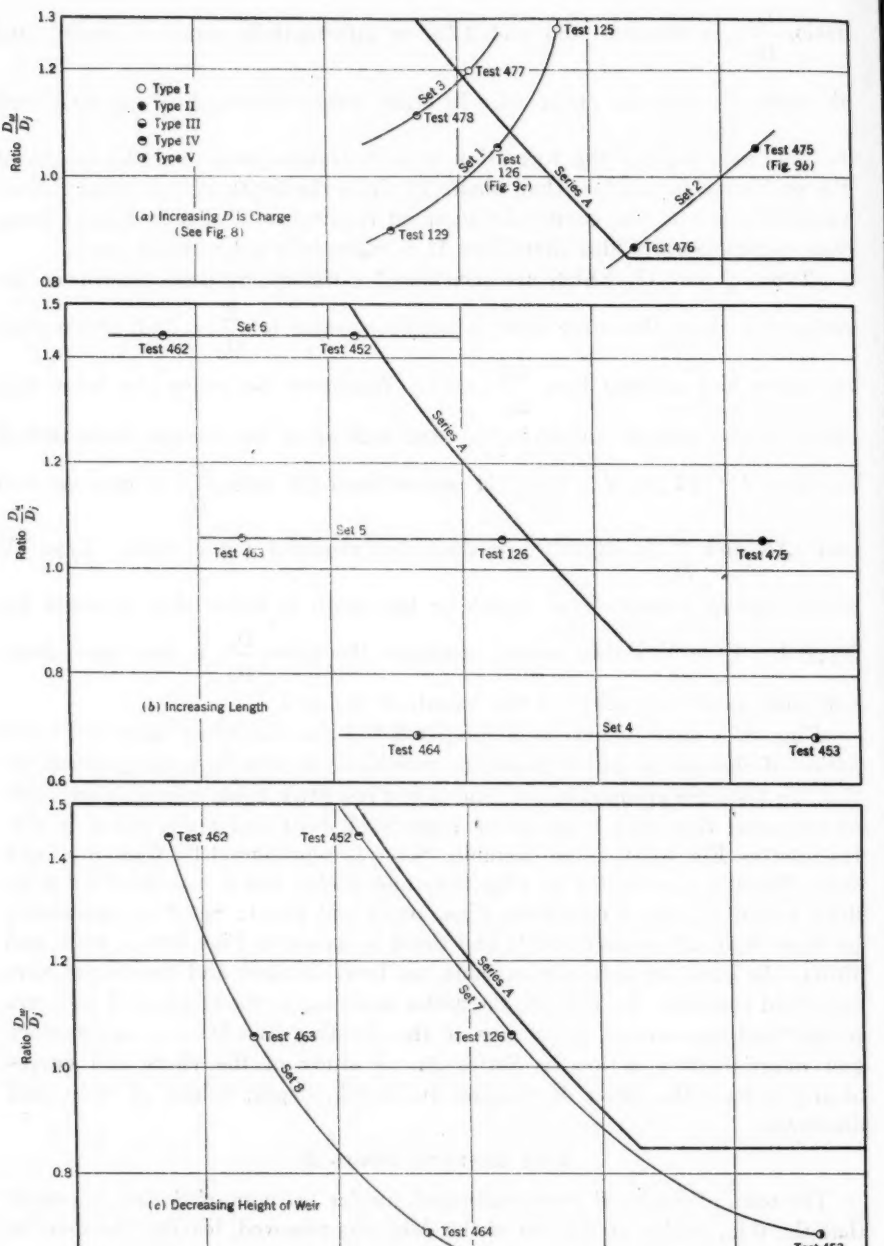


FIG. 11.—RELATION BETWEEN RATIOS, $\frac{L}{D_w}$ AND $\frac{D_w}{D_j}$, SERIES *A*; (*a*) INCREASING DISCHARGE (SEE FIG. 8); (*b*) INCREASING LENGTH (SEE FIG. 9); AND (*c*) DECREASING HEIGHT OF WEIR (SEE FIG. 10).

of $\frac{D_w}{D_j}$ against ratios of $\frac{L}{D_w}$. The solid curve seems to satisfy the data down to the point where $\frac{D_w}{D_j} = 0.8$. This curve is identical with the curve of Series *A*, except that a constant, 1.10, has been subtracted from the exponential function indicating that the basin of Series *B* is considerably more effective than that of Series *A*.

It was considered unnecessary to draw the dividing line between Types I and II and between Types III, IV, and V. It seems logical that the basin

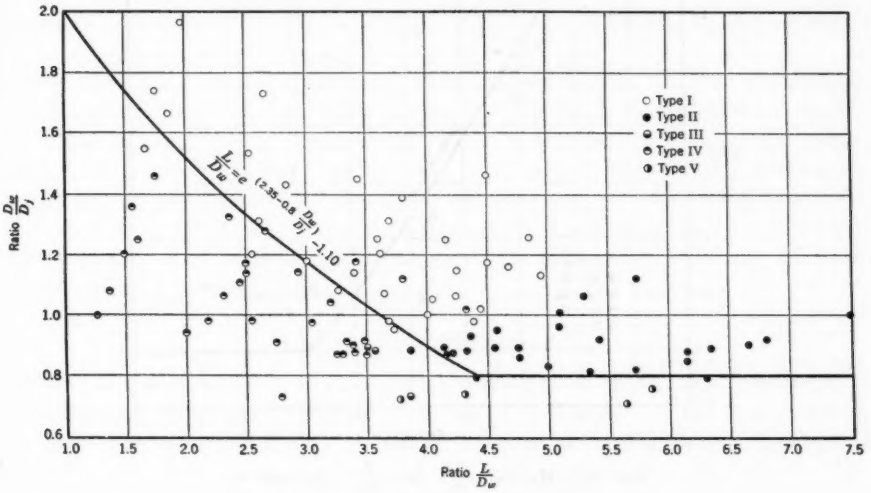


FIG. 12.—RATIOS, $\frac{D_w}{D_j}$ AND $\frac{L}{D_w}$, SERIES *B*.

of Series *B* should be more effective that that of Series *A*, although the magnitude of the difference was rather surprising. With the 6-in. radius of Series *A*, the sheet of incoming water is turned to a horizontal direction with little disturbance, while with the vertical cut-off it impinges with considerable force upon the floor of the bay. A roller or eddy is also created between the vertical face of the dam and the sheet of incoming water.

One other reason for the difference in the effectiveness of the two bays is the method of measuring the length of the bay. In both cases, the measurement is made from the up-stream face of the weir to the intersection of the down-stream slope of the dam with the floor of the flume. This means that with the vertical cut-off, the basin is actually about 0.3 ft. longer than the basin of Series *A* for a given position of the weir. This variation in length accounts for part of the difference in operation, but is not sufficient to account for it entirely. It is obvious that the bay of Series *B* with the vertical cut-off is more effective than that of Series *A*. No attempt has been made to show curves and views of Series *B* such as those shown of Series *A* in Figs. 8, 9, and 10, because they would be similar.

Test Results, Series *C*

The tests of Series *C* and the curve for Series *B* are plotted in Fig. 13 by ratios of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$. The basin of Series *C* was similar to that of Series *B*, except that a triangular strip of moulding was placed on the vertical face, as shown in Fig 5, to approximate a design of the tumble bay proposed[8] by A. F. Meyer, M. Am. Soc. C. E. Only a few tests were run in Series *C*; but

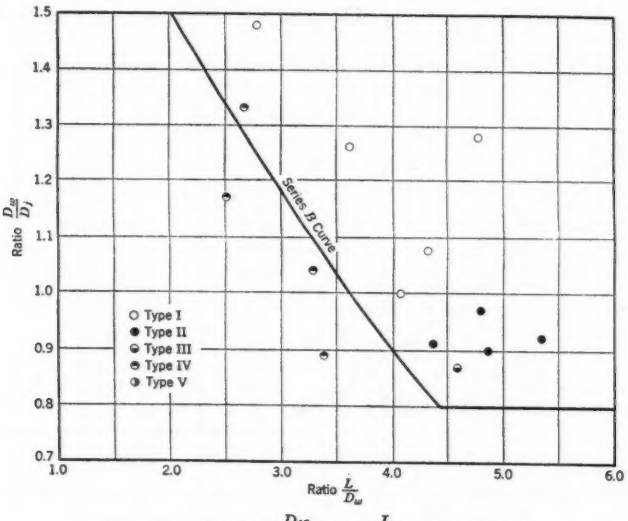


FIG. 13.—RATIOS, $\frac{D_w}{D_j}$ AND $\frac{L}{D_w}$, SERIES *C*.

there is no indication that the curve for this series should be other than that of Series *B*, indicating that the Meyer bay has its effectiveness in the vertical cut-off rather than in the overhanging lip.

Effect of Shape of Dam

The difference between the curves of Series *A* and Series *B* indicates that the shape of the dam has considerable effect upon the action of the stilling basin. The three forms tested in Series *A*, *B*, and *C* (Fig. 5) are entirely too few to set up any definite relationship as to the effect of the shape of the dam. The model dam was built with a down-stream slope of 60° with the horizontal, and there was no opportunity to try other slopes. Differences in action would be expected with differences in slope of the down-stream face of the dam as well as in differences in construction at the toe of the dam. It is to be expected that the steeper the slope, the more effective will be the action of the basin, at least for action of Type I. In the case of a hydraulic-jump action of Type II, it is possible that a flatter slope would allow the jump to form more easily, thus decreasing the length of basin needed for the

[8] "High Velocity Discharge of Overfall Dams and Forms of Spillway Profile," *Engineering News-Record*, April 9, 1925.

complete formation of the jump. Certainly, it may be said that the vertical cut-off principle, as illustrated in Series *B*, is more effective than the bucket of Series *A*. There is need of further research to determine the effect of the shape of the dam on stilling-basin action.

Effect of Shape of Weir

All the tests were run with weirs of similar cross-section, that is, one with a vertical face, horizontal top, and a square corner on the up-stream face. There seems to be no reason to believe, however, that change in the shape of the weir would alter the basic relationship of type of action and the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$. A change in the shape of the weir would change only the value of D_w. It may be assumed that the weirs used in the test had a coefficient of about 3.3, and that a weir with properly designed crest might have a coefficient of 3.7. This would result in a slight decrease of the value of h_w for a given quantity, but as D_w is $W + h_w$ for a given quantity, it would change D_w even less. The result would be an effective height of weir less than that of the test. As the difference in heights of weir produced no differences in the fundamental relations, the differences in shape of weir could not affect the results.

Effect of Submergence of Weir

A few experiments were made in an attempt to establish quantitatively the effect of submerging the weir. The number of tests was toc few to arrive at any definite conclusions. However, it may be said that the immediate effect is to require a greater head on the weir to discharge a given quantity of water. As a result the value of D_w is increased and the ratio, $\frac{D_w}{D_j}$, will also increase with a constant quantity. The other ratio, $\frac{L}{D_w}$, will be decreased slightly but, as D_j is normally much smaller than L, the change in the ratio, $\frac{D_w}{D_j}$, will be of greater magnitude than that of the ratio, $\frac{L}{D_w}$. As a result the depth of the basin is sufficient to handle a greater quantity; hence, the capacity to dissipate energy is increased by submergence of the weir.

Interpretation of Test Results

The conclusions presented herein may be summarized, as follows:

(1) The action of the stilling basin is closely connected to the phenomenon of the hydraulic jump.

(2) The ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, serve satisfactorily to classify the action of a stilling basin into the five types described.

(3) For satisfactory action of a stilling basin, the ratio, $\frac{D_w}{D_j}$, must be greater than 0.85.

(4) For any value of $\frac{D_w}{D_j}$ greater than 0.85, there is a corresponding minimum value of the ratio, $\frac{L}{D_w}$, that determines the minimum proportions of a stilling basin for satisfactory action. As the ratio, $\frac{D_w}{D_j}$, increases the ratio, $\frac{L}{D_w}$, decreases.

(5) Increasing the length of a basin to obtain a ratio of $\frac{L}{D_w}$ greater than this minimum value, does not improve the action. The ratio, $\frac{L}{D_w}$, need never exceed 4.0 or 5.0, depending upon the ratio of $\frac{D_w}{D_j}$, and upon the design of the dam.

Design of Stilling Basins

The design of a stilling basin for any given dam is naturally dependent upon local conditions. A number of basins with varied proportions of depth and length $\left(\text{that is, varying ratios of } \frac{D_w}{D_j} \text{ and } \frac{L}{D_w}\right)$ may be designed to meet a particular height of dam and discharge, as has been discussed herein. The selection of the proportions for a given dam will depend principally upon foundation conditions. Whether or not the basin should be relatively deep or shallow is principally an economic question, the solution of which will depend on the elevation at which satisfactory foundations may be obtained for the dam and the stilling basin. The problem of whether or not the weir is to be submerged will also be principally one of economics; so, also, will be the question of the type of construction to be used at the toe of the dam. Although the vertical cut-off requires a shorter basin, it is accompanied by an additional quantity of concrete in the dam itself. Hence, no general design may be presented to meet all conditions, and satisfactory action will be obtained if the recommended proportions of the basin are used; but the actual design will differ with local conditions.

For design purposes, the solid curves of Figs. 7 and 12 may be plotted on one sheet as a guide to show the minimum ratios of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, for action with dams constructed as in Series *A* and *B*. The required values of D_j for various heights of dam and various depths of discharge over the dam may be taken from a set of curves computed by Equation (1). The writer suggests plotting height of dam against D_j as ordinates for various values of head on the crest. The height of dam is taken as the vertical distance from

the floor of the basin to the crest of the dam. The proportions for basins in which the action will be satisfactory may be readily determined from such curves.

Need of Further Research

In conclusion, the writer wishes to urge further research on this problem. The effect of submergence upon the capacity of a given basin to dissipate energy should be studied further. In studying the effects of the shape of a dam, various slopes as well as various constructions at the toe should be investigated. A series of experiments to determine the resulting scour below the weir for various basins, proportioned from the data included in this paper, would be desirable. Experimentation to determine what part of the basin requires a concrete floor to prevent erosion within the basin should also be made. Such research will lead to additional data of value in establishing a rational method of designing protective devices for overflow dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

BY HERMAN SCHORER, ASSOC. M. AM. SOC. C. E.

HERMAN SCHORER,[54] ASSOC. M. AM. SOC. C. E. (by letter).[54a]—The discussion has brought out many important points which the writer, admittedly, had not treated with sufficient detail. It will not be possible to answer all the questions raised, because the entire subject deserves further theoretical investigation, although the practically important points are now fairly well established.

Mr. Mensch refers to the fundamental work by Professor Love[55] who established the general theory of shells forty years ago. Few engineers would be in a position to apply Professor Love's complete theory directly for practical purposes. Messrs. Templin and Strum, in Fig. 29, present a complete picture of the normal and shearing stress components, the bending and torsional moments, and the load components, that act on a small element in the most general case.

Mr. Mensch doubts the justification of simplifying assumptions made by several investigators. Most engineers will welcome a comparatively simple method, giving results within a few per cent. of the elastic theory. The mathematical physicist will necessarily take a different point of view. Engineering progress, to a large extent, depends on the creation of reliable working tools consistent with the accuracy of the other assumptions made in structural design.

Regarding the bending stresses in symmetrical shells of rotation, symmetrically loaded, Geckeler[56] has presented a detailed discussion of

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna; January, 1932, by Messrs. Paul Bauman and L. J. Larson; February, 1932, by Messrs. C. M. Orr and Edward J. Bednarski; April, 1932, by Messrs. H. C. Boardman, R. L. Templin and R. G. Sturm, and F. Knapp; May, 1932, by Messrs. C. P. Vetter, L. E. Grinter, and Oren Reed; and September, 1932, by Robert E. Glover, Esq.

[54] Chf. Engr., Thebo, Starr & Anderton, San Francisco, Calif.

[54a] Received by the Secretary October 15, 1932.

[55] "The Mathematical Theory of Elasticity," by A. E. H. Love, Third Edition, Cambridge Univ. Press, 1920.

[56] "Zur Theorie der Elastizität flacher rotations-symmetrischer Schalen," by J. W. Geckeler, *Ingenieur-Archiv*, Vol. 1, 1930, p. 255.

his simplified method and that derived by Pasternak,[57] as compared with the fundamental solution by Meissner,[58] the latter strictly conforming to the requirements originally stated by Love. For design purposes, the small deviations can be safely neglected.

The reader is also referred to the publications in the English language by Prescott,[59] Greene,[60] and Coates.[61] The bending stresses in unsymmetrically loaded, cylindrical shells have been treated by Miesel[62] for the case of continuous surface load components, which can be expressed by trigonometric series. Miesel also treats the problem of a full pipe line, inclined at any angle.

All the investigations on bending and torsional shell stresses in closed cylindrical shells, for both symmetrical and unsymmetrical loading, bring out the fact that these stresses are limited to a narrow rim zone, provided the surface loads are distributed over the total shell area in such a manner that a direct stress system is possible. This system of so-called membrane stresses is comparable to the condition of a statically determinate structure.

Mr. Bauman's Fig. 22 clearly illustrates the rapid decrease of the bending deformations with increasing distances from the rim. It is mainly due to the recognition of this point, so typical for structures of the shell type, that their analysis and design can be simplified greatly without any appreciable sacrifice in accuracy.

The development of the Zeiss-Dywidag type of roof, as described by Dischinger,[3] represents a very remarkable step in the application of these principles to open shells for the purpose of wide-span construction over large areas, rectangular or polygonal in plan. Mr. Mensch finds that the stresses given by Dischinger deviate as much as 50% from those obtained by the common theory and claims that the test structures built by the originators were largely over-designed. The writer rather suspects that the common theory used by Mr. Mensch may not represent the actual conditions.

The theory of the Zeiss-Dywidag structures has been verified by numerous model tests made by the designers for the purpose of ascertaining the validity of theoretical simplifications and also at the request of the official building departments entrusted with the authority of guarding the interests of the public and the approval of plans. In addition, independent investigations were made by disinterested experts, among them Professor Kleinlogel. The theory of these structures can now be considered as sufficiently advanced for general application, with due consideration of limiting conditions in the case of discontinuous and concentrated surface loadings. Dischinger[63] has further

[57] "Vereinfachte Berechnung der Biegebeanspruchung in dünnwandigen, kreisrunden Behältern," by P. Pasternak, Verhandlungen des zweiten internationalen Kongresses für Technische Mechanik, Zürich, 1926, p. 427.

[58] "Uber Elastizität und Festigkeit dünner Schalen," by E. Meissner, Vierteljahrsschrift der Naturforsch, Gesellschaft in Zürich, Vol. 60 (1915), p. 23.

[59] "Applied Elasticity," by John Prescott, p. 530, Longmans, Green & Co., Lond., 1924.

[60] "Design of Ellipsoidal Heads for Pressure Vessels," by T. W. Greene, *Transactions*, Am. Soc. M. E., Vol. 51 (1), 1929, p. APM-51-13-137.

[61] "The State of Stress in Full Heads of Pressure Vessels," by W. M. Coates, *Transactions*, Am. Soc. M. E., Vol. 52 (1), 1930, p. APM-52-12-117.

[62] "Ueber die Festigkeit von Kreiszylinderschalen mit nichtachsensymmetrischer Belastung," by Kurt Miesel, *Ingenieur-Archiv*, Vol. 1 (1930), p. 22.

[3] *Proceedings*, Am. Soc. C. E., September, 1931, p. 1002.

[63] "Die Theorie der Vieleckkuppeln," by Franz Dischinger, Wilhelm Ernst & Sohn, Berlin, 1929.

developed the theory of polygonal shells and their relation to inscribed shells of rotation.

Mr. Roop's references to related work give an excellent idea of possible applications of shell design and diaphragm action in other branches of engineering. He mentions, for instance, the bulkheads in a submarine, which correspond to the ring girder of the pipe line. The importance of an accurate knowledge of the stress conditions in this case is evident, not only in order to obtain minimum weights, but also to prevent certain loss of life in case of structural weakness. The same considerations apply to aircraft construction.

Mr. Skytte, Professor Roark, and Mr. Boardman have presented extremely valuable contributions in connection with the condition of stress in a pipe partly filled. As pointed out in the paper,[3] Equations (1), (2), and (3), derived by Bauersfeld, apply only in case the load components can be expressed as continuous trigonometric functions of the angle, u.

When a pipe is partly filled a discontinuous surface load is created, which, in addition to the direct stresses, must also induce bending and torsional stresses in the pipe shell. The fact that equilibrium cannot be established without bending in this case, is clearly brought out by Professor Roark in his Equation (112). Theoretically, it would be necessary to consider all the forces and moments on a small element, as pointed out by Messrs. Templin and Sturm in Fig. 29. Miesel[64] has shown that nineteen differential equations are required for the complete solution of this problem. Their integration, however, presents a mathematical problem of considerable magnitude.

In order to illustrate the general character of the stress condition in a pipe half full, assume the shell to be cut open over its entire length, along the horizontal diameter. Neglecting the dead load, the upper half shell then is free from external loads. With the designations of Fig. 2, the lower half, now comparable to a semi-circular flume, is loaded by the external load components, $X = 0$, $Y = 0$, and $Z = -q\,r\cos(\pi - u)$. The Bauersfeld Equations (1), (2), and (3), applied to the flume section, give the following stress components between the limits, $u = \frac{\pi}{2}$ and $u = \pi$:

$$T_2 = q\,r^2 \cos(\pi - u) \quad \ldots\ldots (213)$$

$$s = q\,r\,x \sin(\pi - u) \quad \ldots\ldots (214)$$

and,

$$T_1 = \frac{q}{8}(L^2 - 4x^2)\cos(\pi - u) \quad \ldots\ldots (215)$$

The shearing stress component, s, for $u = \frac{\pi}{2}$, has a finite value, $s = q\,r\,x$. Since this stress is acting on all four sides of a small edge element, it is necessary to apply an external shearing force of equal magnitude, acting along the edge in the opposite direction, in order to establish equi-

[3] *Proceedings*, Am. Soc. C. E., September, 1931, p. 1002.

[64] "Ueber die Festigkeit von Kreiszylinderschalen mit nichtachsensymmetrischer Belastung," by Kurt Miesel, *Ingenieur-Archiv*, Vol. 1 (1930), p. 32.

librium. This force requires a longitudinal compression member, as shown in Fig. 39(*a*), monolithically connected to the edge of the shell, similar to the tension member in the Zeiss-Dywidag barrel type of roofs. The force

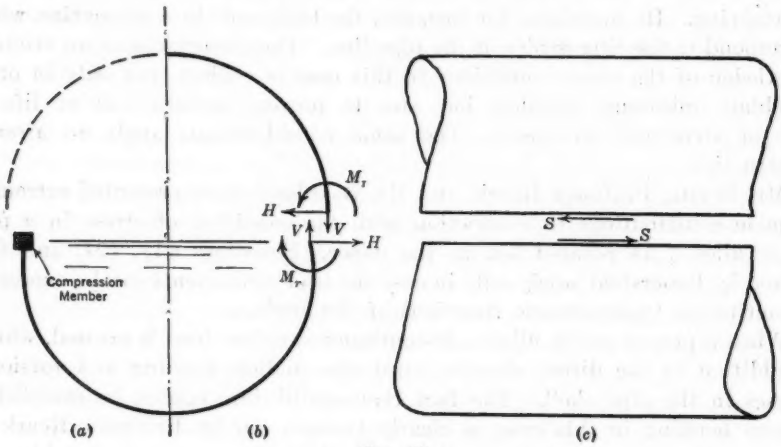


FIG. 39.

in the compression member, designated by P, is equal to the total longitudinal tension in one lower quadrant and is of opposite sign. From Equation (215), by integration between the limits, $u = \frac{\pi}{2}$ and $u = \pi$:

$$P = -\frac{qr}{8}\ (L^2 - 4x^2) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (216)$$

The semi-circular flume then is subject to direct stresses only, except for the secondary stresses caused by the weight and rigidity of the compression member.

This edge member may now be assumed to be replaced by the full shell. Due to the deformation of the loaded lower half shell, a system of edge forces, H, V, and S, and a radial bending moment, M_r, as shown in Fig. 39(*b*), will be required to close the gap and to restore the continuous curvature of the shell. The deformation of a small element also induces longitudinal bending moments, M_l, and corresponding shears, S_l, as shown in Mr. Skytte's Fig. 11. In addition, there are torsional moments acting in radial and longitudinal planes. The values of M_l and S_l, as pointed out by Mr. Skytte, are very small; the same may be said of the torsional moments. This fact is well supported by the careful tests and analysis of the Zeiss-Dywidag roofs. Samsioe's analysis also indicates comparatively small values of M_l. The longitudinal bending stresses in the narrow rim zone of the supporting ring must be considered as a separate stress system, which is of no importance for the condition of partly filled pipes.

Dr. Finsterwalder[65] has shown that the five stress components, T_1, T_2, S, S_r, and M_r (Fig. 11), can then be derived from a stress function, F, similar

[65] "Die Theorie der zylindrischen Schalengewölbe, System Zeiss-Dywidag: Anwendung auf die Grossmarkthalle in Budapest," by Dr. Ing. Ulrich Finsterwalder, International Assoc. for Bridge and Structural Eng., Zurich, 1932.

to Airy's stress function in case of a plane plate. The resulting differential equation is of the eighth order and the integral consists of two damped waves, originating at the loaded edges. Although mainly developed for the purpose of analyzing the barrel type of roofs, Dr. Finsterwalder's method undoubtedly can be adapted for the analysis of any partial filling of a pipe. The writer regrets that limited time and space at present do not permit of entering into Dr. Finsterwalder's method in more detail.

The following results are based on Samsioe's detailed investigation for the special case of a pipe half full. Fig. 40 shows the general character of the direct longitudinal stresses. As shown in Fig. 40(*a*), the longitudinal mem-

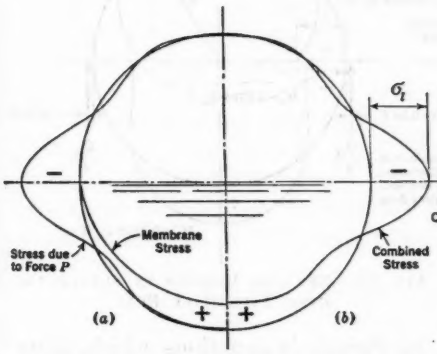


FIG. 40.—PIPE HALF FULL, LONGITUDINAL STRESS.

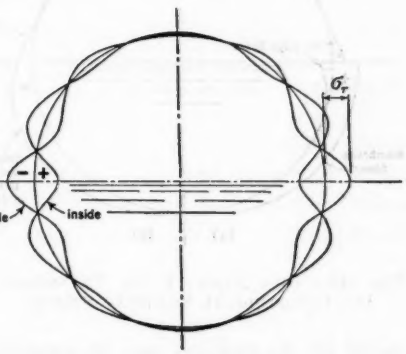


FIG. 41.—PIPE HALF FULL, RADIAL BENDING STRESS.

brane stress (Equation (215)), acts in the lower half of the shell only, and is superimposed by the stress due to the longitudinal force, P (Equation (216)), which is symmetrically distributed about the horizontal diameter in the shape of a damped wave. The relative stress concentration of this force near the horizontal diameter increases with comparatively short spans and thin shells, in which case only a narrow longitudinal zone of the pipe shell acts as a compression member. The combined stress system is shown in Fig. 40(*b*), indicating that a straight-line stress diagram could not represent the actual conditions.

Fig. 41 shows the character of the radial bending stresses, corresponding to the moment, M_r. They are also distributed symmetrically about the horizontal diameter in the form of a damped wave, with a maximum value at the center of the span, decreasing to zero at the rigid supports.

Fig. 42 shows the distribution of the transverse and longitudinal shearing stress, S. This stress (Fig. 42(*a*)), is composed of the shear in the membrane system (Equation (214)), acting in the lower half of the shell only, and the secondary shear due to the edge conditions. The combined value is shown in Fig. 42(*b*). For comparatively short spans and thin shells the combined shear distribution approaches the membrane con-

dition. For large spans and thick shells the distribution becomes identical with that of a full pipe.

For design purposes it is convenient to compare the absolute maximum longitudinal stress, σ_l (Fig. 40), and the absolute maximum radial bending stress, σ_r (Fig. 41), with the maximum beam stress f_L, of a simply supported, full pipe (Equation (13); $w = 0$). The corresponding stress ratios, desig-

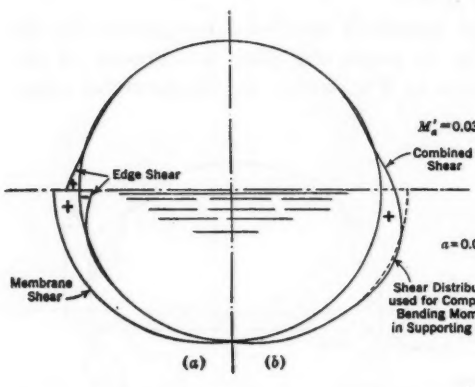


FIG. 42.—PIPE HALF FULL, TRANSVERSE AND LONGITUDINAL SHEARING STRESS.

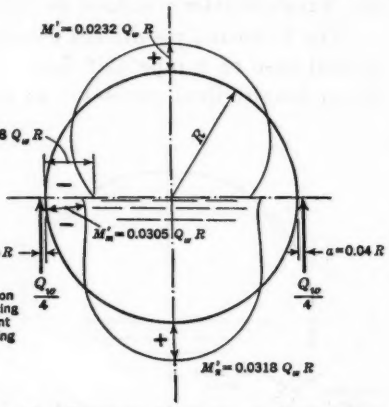


FIG. 43.—BENDING MOMENT IN SUPPORTING RING, PIPE HALF FULL.

nated by n_l and n_r, can be computed by Samsioe's equations which, after a few transformations, become a function of the pure number, k, defined as follows:

$$k = \frac{L}{r}\sqrt{\frac{t}{r}} \qquad \text{.....(217)}$$

In practical cases this characteristic number varies approximately from $k = 0.20$ to $k = 1.20$. A series of values for n_l and n_r has been computed within this range, whereby it was possible to establish the following relations:

$$n_l = \frac{\sigma_l}{f_L} = \frac{1}{\sqrt{k}} \qquad \text{.....(218)}$$

and,

$$n_r = \frac{\sigma_r}{f_L} = \frac{0.31}{\sqrt{k}} \qquad \text{.....(219)}$$

Equation (218) shows that for values of $k \leqq 1$, the half-full condition will give a larger longitudinal stress than in case of a full pipe. The ratio, n_l, remains the same in case of a continuously supported pipe, because both stresses are reduced in the same proportion. The absolute value of the radial bending stress, σ_r, is independent of the type of support.

A comparison of Equations (218) and (219) shows that the maximum radial bending stress is less than one-third the maximum longitudinal stress at the horizontal diameter. In case $k \leqq 1$, the stress, σ_l, will therefore govern the design, but for values of $k \geqq 1$, it will be sufficient to investigate the maximum beam stress of a full pipe.

Applied to the design example, from Equation (217), $k = \frac{720}{60}\sqrt{\frac{0.25}{60}} = 0.775$. From Equation (218), $n_l = \frac{1}{\sqrt{0.775}} = 1.13$, and $n_r = \frac{0.31}{\sqrt{0.775}} = 0.35$.

The maximum beam stress, in inch-pounds, for a full pipe, simply supported, is: $f_L = \frac{62.5 \times 60^2 \times 12}{8 \times 0.25 \times 144} = 9\,400$. The maximum longitudinal stress for a continuously supported pipe, half full, is, therefore, $\sigma_l = \frac{2}{3} \times 9\,400 \times 1.13 = 7\,050$. The maximum radial bending stress is the same as for a simply supported pipe, and becomes, $\sigma_r = 0.35 \times 9\,400 = 3\,300$.

Since the longitudinal stress, σ_l, is zero for a full pipe, it will never occur combined with the rim bending stress, f_{bo} (Equation (27)). It will be permissible, therefore, to allow a comparatively high longitudinal stress under the temporary filling conditions.

The shear distribution for a half-full pipe, as shown in Fig. 42, is also a function of the characteristic number, k. If the value of k is very large, the shear distribution becomes identical with that of a full pipe, giving a bending moment in the supporting ring that is one-half the value found for a full pipe.

As k approaches zero the shear distribution becomes more and more concentrated in the lower half of the shell, as shown in Fig. 42(*b*), drawn for $k = 0.20$. In the extreme case, $k = 0$, the shear follows the law defined by Equation (214), acting between the limits, $u = \frac{\pi}{2}$ and $u = \pi$. By means of Samsioe's equations it would be possible to find the shear distribution for any given value of k, whereby the corresponding bending moment in the supporting ring could then be evaluated. However, the amount of work is considerable and scarcely justified, because the maximum moment is not greatly affected by a variation of k within the practical range.

For design purposes it will be advisable to assume that $k = 0$. This assumption is on the safe side and can be treated similarly to the method shown in the paper under the heading, "Stresses in Supporting Ring." With the designations of Fig. 6, except for Q, the following equations for the bending moments, M'_1 and M'_2, in the first and second quadrants, respectively, have been obtained:

$$M'_1 = \frac{Q_w R}{2\pi}\left[\frac{2r}{\pi R} - \frac{\pi}{4}\left(1 + \frac{a}{R}\right) + \left(\frac{5}{4} - \frac{r}{R} + \frac{a}{R}\right)\cos u\right] \quad \ldots(220)$$

and,

$$M'_2 = \frac{Q_w r}{2\pi}\left[\frac{2r}{\pi R} + \frac{\pi}{4}\left(1 + \frac{a}{R}\right) + \left(\frac{1}{4} + \frac{r}{R} + \frac{a}{R}\right)\cos u - (\pi - u)\sin u\right] \quad \ldots(221)$$

In Equations (220) and (221), Q_w designates the total water load over one support for a full pipe. The moment coefficients for the half-full condition are, therefore, directly comparable with those obtained by Equations (51) and (53), when the weight of the pipe is neglected in the latter equations.

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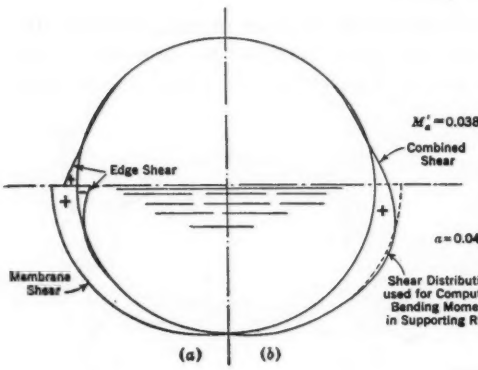

FIG. 42.—PIPE HALF FULL, TRANSVERSE AND LONGITUDINAL SHEARING STRESS.

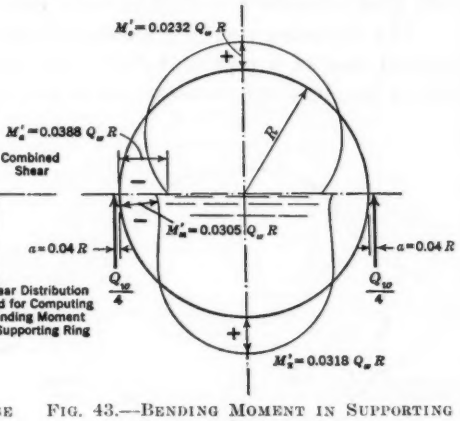

FIG. 43.—BENDING MOMENT IN SUPPORTING RING, PIPE HALF FULL.

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Equation (218) shows that for values of $k \leqq 1$, the half-full condition will give a larger longitudinal stress than in case of a full pipe. The ratio, n_l, remains the same in case of a continuously supported pipe, because both stresses are reduced in the same proportion. The absolute value of the radial bending stress, σ_r, is independent of the type of support.

A comparison of Equations (218) and (219) shows that the maximum radial bending stress is less than one-third the maximum longitudinal stress at the horizontal diameter. In case $k \leqq 1$, the stress, σ_l, will therefore govern the design, but for values of $k \geqq 1$, it will be sufficient to investigate the maximum beam stress of a full pipe.

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The shear distribution for a half-full pipe, as shown in Fig. 42, is also a function of the characteristic number, k. If the value of k is very large, the shear distribution becomes identical with that of a full pipe, giving a bending moment in the supporting ring that is one-half the value found for a full pipe.

As k approaches zero the shear distribution becomes more and more concentrated in the lower half of the shell, as shown in Fig. 42(*b*), drawn for $k = 0.20$. In the extreme case, $k = 0$, the shear follows the law defined by Equation (214), acting between the limits, $u = \frac{\pi}{2}$ and $u = \pi$. By means of Samsioe's equations it would be possible to find the shear distribution for any given value of k, whereby the corresponding bending moment in the supporting ring could then be evaluated. However, the amount of work is considerable and scarcely justified, because the maximum moment is not greatly affected by a variation of k within the practical range.

For design purposes it will be advisable to assume that $k = 0$. This assumption is on the safe side and can be treated similarly to the method shown in the paper under the heading, "Stresses in Supporting Ring." With the designations of Fig. 6, except for Q, the following equations for the bending moments, M'_1 and M'_2, in the first and second quadrants, respectively, have been obtained:

$$M'_1 = \frac{Q_w R}{2\pi}\left[\frac{2r}{\pi R} - \frac{\pi}{4}\left(1 + \frac{a}{R}\right) + \left(\frac{5}{4} - \frac{r}{R} + \frac{a}{R}\right)\cos u\right] \quad \ldots(220)$$

and,

$$M'_2 = \frac{Q_w r}{2\pi}\left[\frac{2r}{\pi R} + \frac{\pi}{4}\left(1 + \frac{a}{R}\right) + \left(\frac{1}{4} + \frac{r}{R} + \frac{a}{R}\right)\cos u - (\pi - u)\sin u\right] \quad \ldots(221)$$

In Equations (220) and (221), Q_w designates the total water load over one support for a full pipe. The moment coefficients for the half-full condition are, therefore, directly comparable with those obtained by Equations (51) and (53), when the weight of the pipe is neglected in the latter equations.

The dead load moment, as found by Equations (51) and (53), must then be added in both cases.

Fig. 43 shows the moment diagram in the supporting ring, for a pipe half full, with $\frac{r}{R} = 0.90$ and $\frac{a}{R} = +0.04$. Fig. 44 gives the moment coefficients for the maximum moments in the function of $\frac{a}{R}$. It will be noted that, for $\frac{a}{R} = +0.04$, the maximum moment in the supporting ring for the half-full condition is 3.88 times the moment value of a full pipe. In case $\frac{a}{R} = 0$, the corresponding ratio is 2.4.

By observation and logic, Mr. Boardman finally arrives at an approximate ratio of three. A similar value has also been estimated by Karlsson. It is

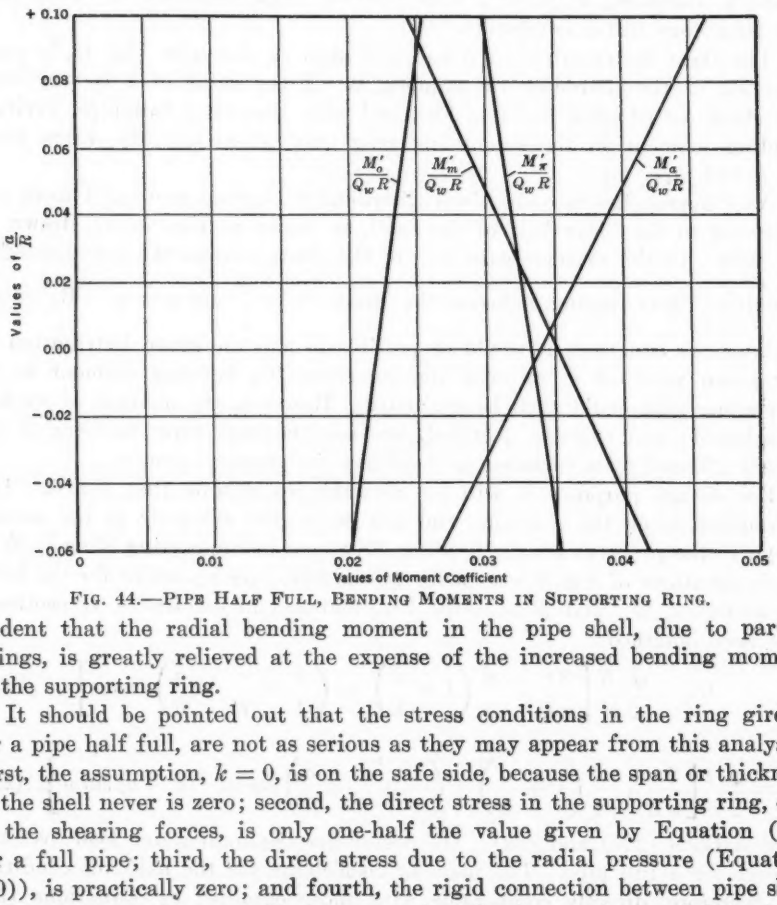

FIG. 44.—PIPE HALF FULL, BENDING MOMENTS IN SUPPORTING RING.

evident that the radial bending moment in the pipe shell, due to partial fillings, is greatly relieved at the expense of the increased bending moment in the supporting ring.

It should be pointed out that the stress conditions in the ring girder, for a pipe half full, are not as serious as they may appear from this analysis: First, the assumption, $k = 0$, is on the safe side, because the span or thickness of the shell never is zero; second, the direct stress in the supporting ring, due to the shearing forces, is only one-half the value given by Equation (57) for a full pipe; third, the direct stress due to the radial pressure (Equation (30)), is practically zero; and fourth, the rigid connection between pipe shell and supporting ring actually increases the effective section modulus of the ring girder; also, the large gusset-plate is placed at the point of maximum

moment. Water-hammer stresses at such a filling are excluded and any inclination of the pipe line will also relieve the bending stresses.

Nevertheless, the stresses in the ring girder for half-full condition should be investigated and compared with the stresses obtained when the pipe is under pressure. In view of the temporary filling conditions and the safe assumptions made in arriving at the moment values, it will be entirely permissible to use stresses close to the elastic limit for this case, although additional resistance to bending can be obtained at small expense. A value, $\frac{a}{R} = 0$, will give more favorable conditions under half-full loading, but higher stresses for full load, than in case $\frac{a}{R} = + 0.04$.

In estimating the possible magnitude of the bending stresses in the pipe walls for half-full condition, Mr. Skytte presents Karlsson's reasoning, as illustrated in Fig. 12. His derivation of the general Equations (59), (60), (61), and (62), represents a forward step. The assumed shear distribution of a pipe half full seems somewhat arbitrary and results in expressions for the maximum radial bending stresses and maximum longitudinal stresses independent of the ratio, $\frac{L}{r}$, as shown in his Equations (67) and (71).

Mr. Skytte's approximate determination of the rim bending stress for an unyielding ring is interesting. By comparing his Fig. 13 with Mr. Bauman's Fig. 22, it will be noted that the approximation is in fairly good agreement with the stress conditions at the rim. In most designs, however, it will be economical to consider the yielding of the ring support in accordance with Equation (27).

The interesting contribution by Mr. D. E. Larson confirms the conclusion that dead load and full water load do not cause circumferential bending in the pipe shell. Mr. Larson also agrees as to the direction of the tangential shearing stresses.

Professor Roark's experimental work is very encouraging and will establish more confidence in the action of supports of the disk type. Since the theoretical treatment of discontinuous surface loads presents considerable mathematical difficulties, the value of model tests can scarcely be underestimated.

Mr. Hanna's suggested derivation of Equation (41) is identical with reversing the assumed algebraic sign of the lever arm, b (Fig. 6), defined by Equation (39). Since a moment is the product of a force and a length, it is evident that such a change necessitates the introduction of the minus sign in his Equation (122). The final result, of course, must be identical.

Mr. Hanna also suggests the investigation of secondary stresses in the supporting ring similar to the methods used for arch dam design. The supports of the ring girder are usually designed as flexible columns or are provided with rocker bearings, so as to allow reasonable movement. In addition, it is well known that the secondary stresses in a semi-circular, fixed arch are generally small, except in comparatively thick arches, in which case, the refinements proposed by Mr. Hanna are justified.

The writer is indebted to Mr. Bauman for his able discussion, especially for his presentation of the elastic deformation of the shell and the load distribution at the rim zone, as shown by Fig. 22, which clearly illustrates the damped wave character of the rim conditions.

Mr. Bauman's derivation of Equations (31), (32), and (33) is of convincing simplicity. The more accurate determination of the tensile stresses in the ring also represents a desirable contribution.

Mr. L. J. Larson presents an interesting comparison of a pipe supported on a horizontal plane, in which a direct stress system can not come into play until large deformations have taken place.

Regarding Mr. Larson's question as to the action of a pipe shell attached to the ring at intervals only, it may be stated that such a design is comparable to the action of a riveted connection. Provided the connections are spaced comparatively close and designed to transmit the corresponding forces in a continuous structure, it is reasonable to assume that the general state of stress would not be greatly affected.

The action of a pipe fitted to the ring in a more or less loose manner would change the assumptions decidedly, because the friction forces can not be depended upon to transmit the shearing stresses, except in case of a shrink fit. If the ring fits rather loosely, then the load will be transmitted mostly by direct bearing on the lower half of the ring. In the extreme case the ring would act as a saddle support, which condition is reached more quickly with a comparatively thick shell, than in case of a thin shell. Fig. 28, presented by Messrs. Templin and Sturm, is well adapted to illustrate this point.

Mr. Larson's suggestion as to the use of the dished heads of horizontal tanks as supporting disks is very timely and of great practical importance. The writer has not made any analysis of this possibility, but it is evident that a direct stress system is possible with tangential column supports attached at the center of the dished heads.

As a first approximation, the two end reactions could then be considered as a concentrated force, applied to a point in the direction of a plane of an infinite flat plate, which problem is treated by Prescott.[66] The reaction must be applied over a sufficiently large area, so as to keep the local stresses within safe limits. For this purpose the central area of the dished head may require a gradual increase in thickness. In many cases the flange of a manhole or outlet, located at this point, could be used to transmit the forces.

Mr. Orr finds that the bending stresses in the rim zone can be safely neglected, although he does not question their existence and considerable magnitude. The writer is unable to agree with Mr. Orr on this point, in spite of the fact that standard specifications may not require their consideration. Working stresses close to the elastic limit, especially when accompanied with frequent stress reversal, sooner or later may lead to failures.

Mr. Orr's statement regarding the design of pressure containers, is decidedly in error. The proposed changes in the A. S. M. E. Code for Pressure Vessels, especially the specifications for ellipsoidal heads, are a direct attempt to eliminate or minimize the high stress concentration in the rim zone.

[66] "Applied Elasticity," by John Prescott, Longmans Green & Co., Lond., 1924, p. 375.

Mr. Bednarski gives an interesting account of the experience on the pipe line of the Los Angeles Aqueduct, supported on piers of the conventional type. In most practical cases the advantages mentioned by Mr. Bednarski are more than offset by other factors. Although it is quite true that the pipe line, when continuously kept under pressure, would not require any rigid ring supports to maintain its circular shape, it should be pointed out that such ideal operating conditions are seldom realized.

The large deformations at partial filling must invite failure or, at least, cause serious leakage, requiring frequent caulking and replacement of protective paint coatings. A pipe line supported on piers, with comparatively short spans, must necessarily follow the natural contours of the ground very closely, whereas long-span construction permits the choice of a shorter route, which may result in considerable savings in material, together with smaller friction losses.

Mr. Boardman's excellent contribution is of decided value, because it raises important questions in connection with partial filling, already answered in connection with Mr. Skytte's discussion and that of Professor Roark. Mr. Boardman's explanation of the small deformations for discontinuous loadings, as observed on his model, and his reasoning regarding the re-distribution of transverse shearing stresses, closely coincide with the approximation already advanced by Karlsson.[2] Instead of the concentrated shearing forces as indicated by Mr. Boardman in Fig. 26, Karlsson assumed a continuous distribution proportional to sin 2 u, also used by Mr. Skytte.

By letter dated June 6, 1932, Mr. Boardman has informed the writer of an interesting test on the tank illustrated in Fig. 45, which is a convincing application of theory to practice. The tank is 12 ft. 0 in. in diameter, 35 ft. 0 in. long, and both shell and heads are made of $\frac{1}{4}$-in. plates butt-welded together, each being stiffened by two horizontal channels, 9 in. by 13.4 lb. welded on the inside. According to Mr. Boardman, this tank, supported entirely on the four vertical angles, two on each head (see Fig. 45), was filled with water and then emptied; no deformations were noticeable to the eye.

Equation (217) gives a characteristic number, $k = 0.343$, for this tank. The maximum longitudinal stress for half-full condition, from Equation (218), is $\sigma_l = 3\,180$ lb. per sq. in., and the maximum bending stress in the pipe walls, from Equation (219), is $\sigma_r = 1\,670$ lb. per sq. in. The magnitude of these stresses sufficiently explains the small deformations observed by Mr. Boardman.

Messrs. Templin and Sturm are to be congratulated for presenting their extremely valuable investigation on the pipe-line problem. All their deductions are based on extensive experimental data, combined with theoretical considerations. It is gratifying to note that, beginning with the tests made on the Santeetlah Pipe Line, Messrs. Templin and Sturm finally arrive at the type of support illustrated in Fig. 32.

Regarding the relative location of the neutral axis of the ring, the writer quite agrees that the integral action of pipe shell, stiffener, and column sup-

[2] *Proceedings*, Am. Soc. C. E., September 1931, p. 1000.

port requires certain assumptions as to the proper value of a (Fig. 6). In case of the Glines Canyon pipe line the reactions are transmitted to rocker bearings, with the bottom plate carefully adjusted and grouted in place, whereby the direction and location of the reaction are fairly well assured.

In a recent design of ring supports for the horizontal type of gas-holders and liquid butane storage tanks, as illustrated in Fig. 46, a soft lead plate was inserted between two steel bearing-plates, permitting plastic deformations and re-adjustment in case of eccentric loadings. The rigid connection between ring and shell was not taken into consideration for the ring design, but this assumption is evidently on the safe side. The typical distortion curves of unstiffened pipe shells on saddle supports, with comparatively short spans, as shown in Fig. 28, will serve as an instructive guide for designs of this type.

Equation (170), based on simplifying assumptions, is identical with Equation (9). The same qualifying considerations for partial fillings, as brought out by Mr. Boardman and already answered, also hold for the analysis of the ring supports of the type investigated by Messrs. Templin and Sturm. The writer would welcome a publication of any tests made for partial fillings.

The inclined strut undoubtedly gives a better gusset-plate connection, but requires the addition of the horizontal tension member. However, the choice can be safely left to practical considerations in each case.

Mr. Knapp's reference to the previous paper by Professor Thoma, also referred to by Professor Roark and Messrs. Templin and Sturm, is appreciated.

The writer regrets that only an abstract of the thesis by Dr. Abdank[67] is available, although a complete report on the extensive model tests, made in connection with the Shannon River penstock, would undoubtedly be of general interest.

A comparison of the maximum longitudinal stresses, as given by Mr. Knapp's Equation (186), with those obtained by Samsioe's equations, for the same case of loading, does not indicate any reasonable agreement. The large discrepancy is probably due to the fact that Equation (186) does not contain the ratio, $\frac{L}{r}$, which certainly must enter into any general expression.

Mr. Vetter's derivation of Equations (187), (188), and (189), is convincing. His proof that the ordinary beam formulas, in case of a full pipe, also apply to structures of the cantilever type, is a desirable contribution. Mr. Vetter applies the theory of string polygons to a free circular ring and shows in a very original manner that the pressure line for a full liquid load is a circle, whereby the stability of the ring under direct stresses is clearly demonstrated.

Professor Grinter's discussion is gratifying and raises important practical points in connection with the proper design of storage tanks. Experience will always confirm his excellent statement that the important consideration

[67] "Berechnung ganz oder teilweise gefüllter, frei tragender, dünnwändiger Rohrleitungen mit beliebig geneigter Achse," by R. Abdank, *Die Bautechnik*, June 19, 1931, p. 420.

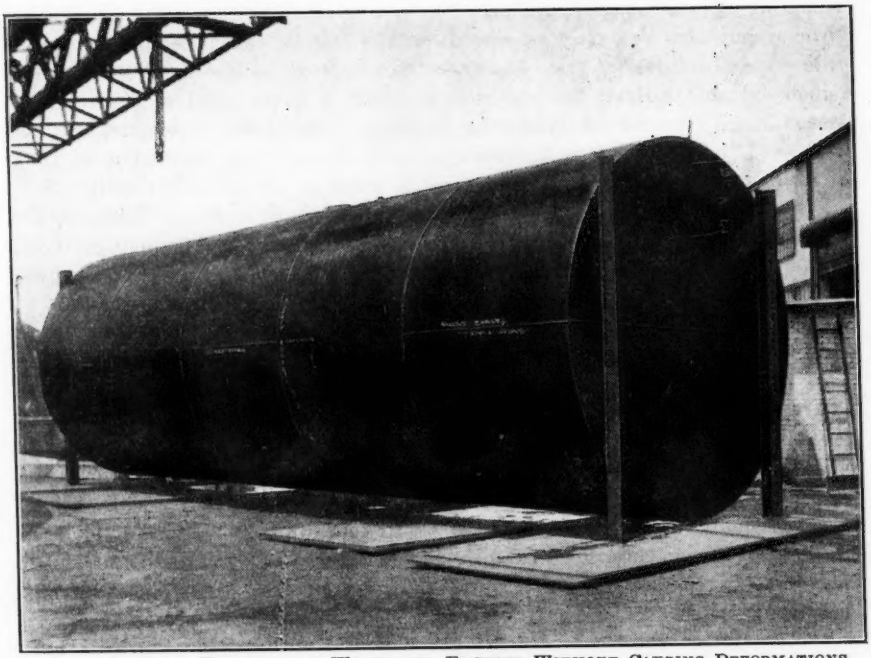

FIG. 45.—TANK FILLED WITH WATER AND EMPTIED WITHOUT CAUSING DEFORMATIONS NOTICEABLE TO THE EYE.

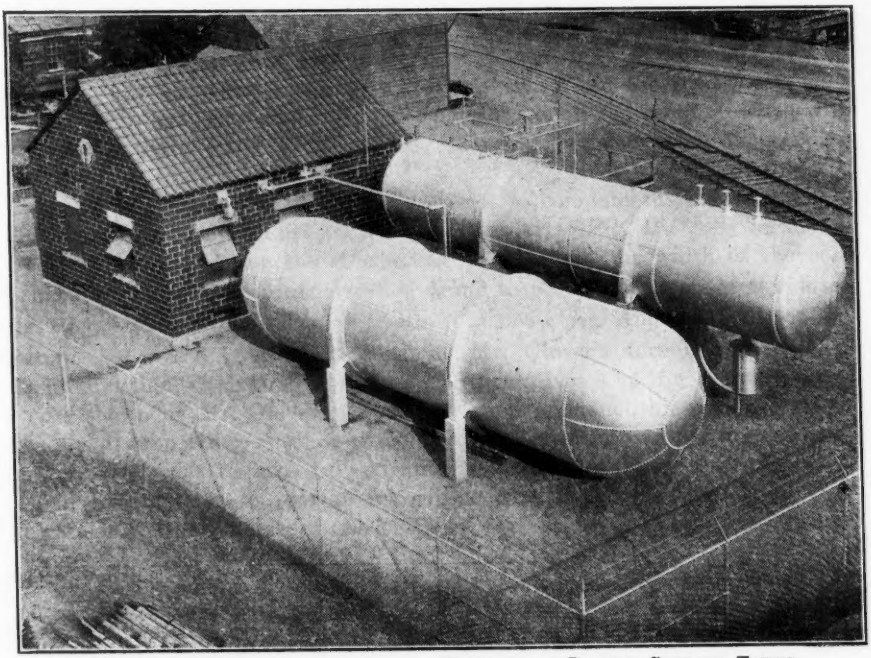

FIG. 46.—HORIZONTAL TYPE OF GAS-HOLDER AND BUTANE STORAGE TANKS.

is to produce a structure of such stiffness that any possible change in shape will not produce leakage at the joints. Professor Grinter's warning regarding careless application of formularization is very timely. The careful designer can scarcely avoid a study of the original publications on shells. For this purpose a reasonable amount of mathematical training must necessarily be considered as a fundamental requirement.

Mr. Reed's description of some of the more recent Swedish pipe lines is well adapted to establish further confidence in supports of the ring type, which evidently has become common practice in Sweden for economical reasons.

Mr. Glover's discussion is of decided value, because it raises the question as to what part of the pipe shell can be considered as a flange of the ring girder, when monolithic action is assured. The same problem has also been discussed by Messrs. Templin and Sturm, who suggest that, based on tests of a flat plate, stiffened with two channels back to back, about eight times the thickness of the plate outside each row of bolts can be considered as acting monolithically with the stiffener.

Mr. Glover's analysis is based on the stress condition in an infinitely large flat plate, loaded by forces in its plane along a straight line, the magnitude of these forces varying as the ordinate of a sine curve. Assuming that the closed shell acts in a manner similar to a plate, Mr. Glover then arrives at a total effective width of the shell, $\beta = 0.383\, r$ (Equation (212)), whereas Messrs. Templin and Sturm suggest $\beta = 16\, t$, when both sides of the ring are considered.

The writer is unable to agree with either suggestion. Evidently, the effective width in the case of a shell is a function of both r and t. This fact may be observed roughly by compressing rubber tubes of different dimensions at the two opposite points of a diameter. From the theory of shells it may be concluded that a wave-shaped load around the circumference will not change the effective width greatly, unless the number of waves becomes fairly large.

As an approximation it will be sufficient to compute the effective width of a shrink fit on a thin cylindrical shell, similar to Fig. 4(*a*). By means of Geckeler's[8] equation for the ring stress in the shell, the width of the corresponding zone of uniform stress is found to be $0.78\sqrt{r\,t}$ on each side of the ring, giving a total effective width, $\beta = 1.56\sqrt{r\,t}$. Applied to the design example, $\beta = 1.56\sqrt{60 \times 0.25} = 6.0$ in., Mr. Glover's formula gives about 23 in. and that of Messrs. Templin and Sturm gives 4 in. for the same case.

As shown by Mr. Glover, the consideration of monolithic action between shell and supporting ring results in reduced bending stresses in the ring girder. In order to take full advantage of the imaginary shell flange, it would be advisable to provide a corresponding flange of equal width and thickness on the outside of the stiffener web, so as to obtain true I-beam action.

The writer wishes to express his thanks to all those who have participated in the discussion of this paper.

[8] *Proceedings*, Am. Soc. C. E., September, 1931, p. 1008.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SOIL MECHANICS RESEARCH

Discussion

BY MESSRS. F. KÖGLER, AND GLENNON GILBOY

PROF. DR.-ING. F. KÖGLER,[117] (by letter).[117a]—Under the heading, "Research in Soil Engineering (Foundations)," Professor Gilboy states that his studies have shown that the settlement under a given unit load not too near the ultimate is practically independent of the size of the loaded area.

An extended series of tests performed under the writer's direction in the Soils Laboratory of the Bergakademie, in Freiberg, Germany, yielded valuable information on the subject as early as 1924. Figs. 49 and 50 show the relations between settlement and diameter of loaded area as derived from these studies. The laboratory tests agreed, in the main, with tests performed in the field.[118]

The first part of this research program was devoted to careful measurements of the distribution of vertical stress under loaded areas.[119] Later, the settlement characteristics of areas of similar shapes but of different sizes were studied. The curves of Fig. 49 were obtained from the results of a series of such tests on seven different circular areas with diameters ranging from 8 cm. to 100 cm.[120] The researches were next extended to include studies of the settlement characteristics and ultimate bearing capacities of areas of equal size but of different shapes.[118]

The shape of the curves in Figs. 49 and 50 may at first appear rather peculiar; but it can easily be explained. The settlement of a loaded area is due to two factors: First, to the compression (elastic and non-elastic), of the

NOTE.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: December, 1931, by Messrs. J. C. Meem, and H. de B. Parsons; January, 1932, by Messrs. Jacob Feld, and John R. Jahn; February, 1932, by Messrs. C. H. Eiffert, A. A. Eremin, F. N. Menefee, and E. G. Walker; April, 1932, by Messrs. D. P. Krynine, J. Stuart Crandall, and Frank Alwyn Marston; and May, 1932, by Messrs. George Paaswell, John H. Griffith, and H. E. Gruner.

[117] Director, Foundations Laboratory, Bergakademie, Freiberg, Saxony, Germany.

[117a] Received by the Secretary April 14, 1932.

[118] Kögler, "Ueber Baugrund-Probebelastungen," *Die Bautechnik*, 1931, H. 24.

[119] Kögler-Scheidig, "Druckverteilung im Baugrunde," *Die Bautechnik*, 1927, H. 29 und 31; 1928, H. 15 und 17; 1929, H. 18 und 52.

[120] E. W. Goerner, Assoc. M. Am. Soc. C. E., "Der Einfluss der Flächengrösse auf die Eisenkung von Gründungskörpern," Dissertation, Freiberg, 1928, *Geologie und Bauwesen*, Heft 3, 1932.

soil; and, second, to the displacement of the soil sidewise and upward. The latter factor is especially marked in the case of loads applied to the surface of the soil over small areas; the smaller the area, the more marked is the influence of lateral displacement.

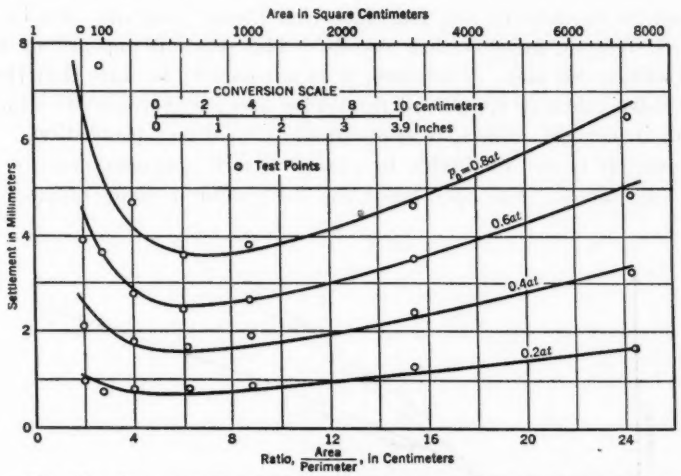

FIG. 49.—RELATION BETWEEN SETTLEMENT AND LOADED AREA, OF SAND.

This relation is reflected in the appearance of the left branches of the curves in Fig. 49. An understanding of this action is essential to the proper interpretation of the results of loading tests on small areas. The larger

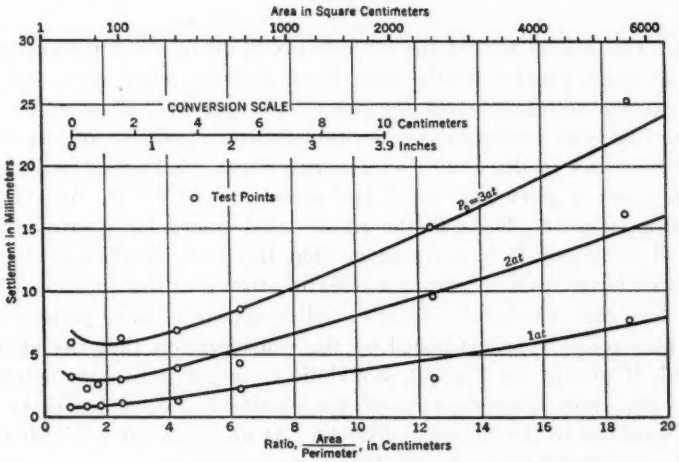

FIG. 50.—RELATION BETWEEN SETTLEMENT AND LOADED AREA, IN CLAY.

the area and the greater its depth, the smaller becomes the influence of this factor. Even so, however, it may still be appreciable if the soil is of a character offering relatively low resistance to lateral displacement.

With regard to the direct compression of the soil, theoretical considerations lead to the conclusion that the settlement due to this factor is proportional to the diameter of the loaded area. The writer's tests confirm this relation, as indicated by the right branches of the curves in Fig. 49. Tests performed in the field by Dr.-Ing. Heinrich Press[121] and the tests by Professor W. S. Housel, gave similar results. The relation appears to hold for sand as well as for clay. Therefore, it is not correct to state that the settlement is independent of the size of the loaded area; it increases with increasing size, and, in certain ranges, is approximately in direct proportion.

However, it is not justifiable to assume that this proportionality extends to very large areas. For such areas the settlement becomes correspondingly

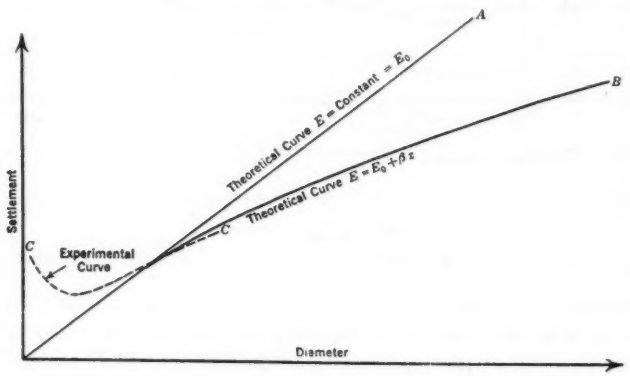

FIG. 51.

smaller. The reason is that the stresses produced in the soil by a large area extend to much greater depths than those under a small area, and at such depths the soil is denser and the lateral resistance is greater.[122] If it is assumed that the coefficient of compressibility (modulus of elasticity) of the soil increases in direct proportion with depth, theoretical analysis of the problem gives a curve of the type, shown at *B*,[123] Fig. 51. Even this assumption, however, leads to the results that under large areas the settlement will be large. It is unfortunate that there are insufficient data on the behavior of large areas to permit a definite answer to the question.

The fact that Professor Gilboy's studies appear to have yielded different results may possibly be explained by the consideration that the areas which he tested, if plotted on Fig. 49, would lie near the minimum points of the curves. An exact interpretation of his results would, under these circumstances, conform to the curves of Fig. 49. At all events, it is highly desirable to obtain settlement data on very large areas.

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[121] *Die Bautechnik*, September 26, 1930.

[122] Scheidig, "Versuche über die Formänderung von Sand und ihre Anwendung auf die Setzungsanalyse von Bauwerken," Wien, 1931 (unpublished).

[123] Aichhorn, "Ueber Zusammendrückung des Bodens infolge örtlicher Belastung," Dissertation, Freiberg, 1931, *Geologie und Bauwesen*, 1932, H. 1.

GLENNON GILBOY,[124] JUN. AM. SOC. C. E. (by letter).[124a]—The number of discussions contributed to this paper is indicative of the rather widespread interest now being taken in soil mechanics. In spite of the difficult nature of the problems involved, there appears to be a growing sentiment in favor of continuing the attack from all angles, thereby adding slowly but steadily to the somewhat meager fund of knowledge available. The information thus far accumulated affords a clearer understanding of the behavior of soil as a construction material than has hitherto been possible; and persevering study is certain to produce comparable results in the future.

Mr. Meem's idea that bearing values should be stated in terms of settlement is quite logical. However, the writer is not optimistic with regard to the possibility of reducing the problem of settlement to a relatively simple classification as outlined by Mr. Meem. Furthermore, there is not much hope that loading tests, even on the broad scale advised by Mr. Meem, will furnish adequate information. To mention only one difficulty, the time element involved in the settlement of foundations under-laid by clay makes it difficult, perhaps impossible, to predict the behavior of a large building over a long period of years from no other data than the results of a test on a limited area lasting a relatively short time.

The apparatus suggested for lateral pressure tests is particularly interesting because, some time ago, the writer outlined a machine identical in principle, although on a much smaller scale. The writer is in full agreement with Mr. Meem regarding the importance of tests made on this basis. Their performance is impeded only by lack of funds.

In answer to Professor Parsons' question, the coefficient of permeability as used in Equation (2) is not a constant; for a given soil it varies not only with temperature, but also with the density of the soil mass.

Mr. Feld's misunderstanding of the definition of water content is probably due to a poor choice of terms on the writer's part. Instead of "weight of water per unit weight of dry soil," it might have been clearer to say, "weight of water per unit weight of solid." For example, if the original weight of a sample of soil is W, and the weight of the same sample "bone dry" is W_o, the water content of the original sample, w, is, $w = \frac{W - W_o}{W_o}$.

The objection by Mr. Feld to the lack of homogeneity of Equation (1) is sound. Here, again, a looseness in statement is apparent. To be strictly correct, the quantity, $V_1 - V_o$, should be multiplied by the weight of water per unit volume. Since, however, this is unity in the centimeter-gramme-seconds system, one easily falls into the habit of forgetting its existence in routine laboratory work.

There is still considerable controversy concerning the effect of the size of the loaded area on the magnitude of settlement. Although the tests with which the writer has had to deal justify no other conclusion than the one stated, it is possible that the range of areas involved was not sufficiently great

[124] Associate Prof. of Soil Mechanics, Mass. Inst. Tech., Cambridge, Mass.
[124a] Received by the Secretary September 27, 1932.

to show the true situation. Extended series of tests performed in Freiberg under the direction of Professor Kögler indicate that the settlement on sand increases with the size of the area. On the other hand, this increase can scarcely be unlimited. If it were, large and heavy buildings founded on deep sand deposits should undergo considerable settlement; but this does not seem to be the case. A number of fairly heavy buildings on Lower Manhattan Island are founded on deposits of sand, frequently rather loose in structure, extending to depths of 100 ft. and more; yet if the writer's information is correct, settlements in this locality are relatively small.

To quote some figures, one series of the Freiberg tests on different areas under a pressure of 0.8 ton per sq. ft., gives a curve of which the asymptote may be represented approximately by the equation, $S = 0.026\,R$, in which, S is the settlement and R, the ratio of area to perimeter. This extension leads to the result that a building founded on a mat of 100 by 200 ft., with the very light load of less than 1 ton per sq. ft., should settle about 10 in.; or that a subway 50 ft. wide, with the same light load, should settle about 8 in. Such conclusions are manifestly absurd.

Just where the discrepancy lies, the writer is not at present prepared to state. It is evident, at any rate, that considerable work remains to be done in order to bring out the true relations between the results of loading tests and the behavior of actual structures, even when operating with a relatively simple material, such as cohesionless sand.

Mr. Jahn's pertinent comments on the hydrometer method illustrate the difficulties involved in working out a procedure for wet mechanical analysis. Certain of the faults enumerated can be minimized by the use of proper corrections. It is true, as Mr. Jahn intimates, that even with all possible corrections taken into account the method still remains approximate; but the margin of error is small enough so that the method fulfills the usual requirements of practice. Its many advantages, including the low cost of equipment, the ease of operation, and the short time consumed, make it a useful adjunct to routine laboratory work.

The writer was favorably impressed by Mr. Jahn's scheme of using a balanced immersion cylinder for this work, although it seems probable that the apparatus might be expensive. The scale-beam would have to be extremely sensitive, and the weights accurate, on account of the small changes in the uplift of the soil suspension. In addition, the temperature of the outfit would have to be kept constant, in order to eliminate changes in uplift due to changes in density of the water.

Mr. Eiffert's clear description of the field work at the Germantown Dam requires no comment. It may be useful, however, to clarify the meaning of the term, "percentage consolidation." To say that a soil is 25% consolidated means nothing, of itself, with regard to solidity. It merely means that of the total quantity of water that must eventually be forced out of the soil mass by the pressure to which it is subjected, one-fourth has already been forced out. In order to determine the solidity of the material, its angle of internal friction and the magnitude of the pressure acting upon it must also be known. For example, if a soil having the characteristics of the Ger-

mantown core material was subjected to a pressure of 1 ton per sq. ft., it would develop, if 100% consolidated, a compressive strength of about 1.7 tons per sq. ft. On the other hand, if the same soil were under a pressure of 4 tons per sq. ft., but only 25% consolidated, its strength would still be 1.7 tons per sq. ft. Thus, it is evident that the degree of consolidation alone is not sufficient to define the characteristics of the material.

With regard to the slow rate of consolidation of the Miami cores, it is the writer's opinion that this is a favorable feature, much to be preferred to the situation arising in a core which has been made coarser purposely to facilitate rapid consolidation. If the core is stable at a low degree of consolidation, additional hardening merely adds strength where it is not needed. Rapid and complete consolidation is, therefore, neither necessary nor desirable. Conversely, a core that consolidates slowly has, in addition to its high degree of water-tightness, the advantages that the re-adjustments of the entire dam accompanying consolidation are very gradual, and that there is no danger that drying and cracking of the core, in the manner described by Mr. Eiffert, will take place.

Mr. Eremin's remarks on colloids touch on one of the many effects that complicate the physical and chemical properties of soils. In fact, it is on account of these complications that direct methods of testing appear much more promising than attempts to infer the physical and mechanical properties from detailed technical analyses.

Professor Menefee has brought out some interesting points concerning the rupture of materials under compression. It is true that specimens of concrete and of soil frequently fail due to lateral tension, the failure being accompanied by vertical cracks; but this action is not the reason why true shear failures do not occur on planes making an angle of 45° with the principal planes. Rupture in shear takes place along a given plane when the obliquity of the resultant stress on that plane equals the angle of internal friction of the material. Therefore, if a specimen fails in shear, it will fail along planes upon which the obliquity of stress is a maximum. These planes are not necessarily planes of maximum shear intensity. In a material, such as steel, having a low angle of internal friction, the planes of maximum obliquity and the planes of maximum shear practically coincide; but this condition no longer prevails when the angle of internal friction has a value appreciably greater than zero. The greater the angle of internal friction, the greater is the angular displacement between these two sets of planes.

Referring again to Mohr's diagram in the original paper (Fig. 9), the positions of the planes of maximum obliquity are represented by the two radii drawn to the points of tangency. Suppose that the angle of internal friction of the soil specimen is 26°; then the angle, α, is 64 degrees. This is twice the angle between each plane of maximum obliquity and the plane of minor principal stress. Therefore, shear failure of the specimen will take place along planes making an angle of 32° with the vertical, in spite of the fact that the maximum intensity of shearing stress occurs on 45° planes. It is thus evident that Professor Menefee's observation in connection with Fig. 19 that "actual tests show that the failures in earth cylinders and con-

crete as well are along planes of greater slope than 45°," substantiates the theoretical concept.

Mr. Walker is correct in stating that the changes in consistency of clay as the water content decreases are not all marked by well-defined points of division. The shrinkage limit is an easily recognized boundary, but the plastic and liquid limits—especially the latter— are really transition states. It is precisely for this reason that standardized methods of determination of these limits are essential.

Mr. Walker suggests the correlation of moisture content with the force required to produce some agreed distortion of the sample. This is, in effect, the basis of the liquid limit determination. The agreed distortion is represented by the flowing together of the two halves of the sample over a specified distance. The force is measured by the number of impacts required to produce this distortion. When the number of impacts required equals an agreed number—say, 25—the water content of the sample is said to correspond to the liquid limit. To this extent the definition is arbitrary; but, as long as the adopted standard is adhered to, the results will be consistent, and the values will afford a definite measure of the character of the material.

It should also be noted that any elaborate method of determining the limits, or of setting up comparable standards, would defeat its own purpose. As the writer has pointed out, the chief advantage of the limit determinations is their simplicity. As soon as they become difficult to handle, their utility ceases.

Mr. Walker's unfavorable opinion of the shrinkage limit equation undoubtedly arises from the same difficulty that misled Mr. Feld. If Equation (1) is written:

$$S = \frac{(W_1 - W_0) - \rho\,(V_1 - V_0)}{W_0}$$

in which, ρ is the weight of water per unit volume, the obscurity disappears. The shrinkage limit is, by definition, the weight of water in the sample at the limit, divided by the dry weight of the sample. The quantity, $W_1 - W_0$, is the total weight of water in the saturated sample at the outset. The quantity, $\rho\,(V_1 - V_0)$, is the weight of water that disappeared from the sample as it dried from its original state to the shrinkage limit. The difference must be the weight of water that the sample contained at the shrinkage limit. Dividing this by the dry weight gives the desired value.

It is advisable, as Mr. Walker suggests, to calibrate the capillary permeameter before using it on an extended program of tests. However, the writer adheres to his original opinion that the apparently large variation in the coefficient, F, does not prevent the use of the uncalibrated apparatus for approximate measurements. If one could be absolutely certain of seepage calculations within 50% or even 100%, it would represent a fair degree of accuracy. For example, the permeability of the apparently homogeneous core of the Germantown Dam, determined from direct tests on undisturbed samples, varied from 20×10^{-7} to 400×10^{-7} cm. per min. In heterogeneous natural deposits the variations are many times greater. Hence, an approximate measurement that is within 50% of the truth is by no means worthless.

In scientific research, a high degree of accuracy is essential; but in the practical field, the most elaborate computations frequently can do no more than define the general region within which the true solution lies.

The remark by Mr. Walker concerning the end effect in the retaining-wall apparatus brings out one of the main difficulties associated with devices of this type. The effect is not quite as marked as Mr. Walker states. Although the bin is 10 ft. deep, the model wall is only 7 ft. high, so that the ratio of depth to width of restrained earth is 1 to 2. In addition, the depths actually used in the tests were somewhat less than 7 ft., thus giving an even more favorable ratio. A further attempt was made to minimize the side friction by greasing the wall before placing the fill. Therefore, while certain end effects were always apparent, it is believed that they were not of sufficient magnitude to invalidate the results.

Professor Krynine's bibliography and synopsis of the investigations pursued by many scientists should be useful to those unfamiliar with the field of soil mechanics as an indication of the time and effort expended on the study of various important problems.

Mr. Crandall's remarks on the present unsatisfactory methods of soil classification are most pertinent. The writer did not touch on this subject because of the lack of anything concrete to offer. The problems involved in an attempt to arrive at a rational basis of classification are rather numerous. In the first place, the scheme of classification would have to be such as to meet the requirements of the particular purpose for which the standards are intended. A system satisfactory to an agronomist might be misleading to a foundation engineer. The sub-divisions should be based on variations in those physical properties of the soil that have a direct bearing on the behavior of the material from an engineering standpoint. There should be a sufficient number of sub-divisions so that any known soil would find a definite place, but not so many as to make the system cumbersome to use. The criteria of allocating soils to their proper sub-divisions should be definite, simple, and readily reproducible. Any tests required for applying these criteria should be of such an elementary nature as to be readily performed in the field, at least for a rough preliminary classification. The system as a whole should be of such a sensible and straightforward nature as to appeal immediately to the practical man. Finally would come the not inconsiderable task of educating engineers, architects, contractors, and drillmen in the essentials of the system, in order to make its use sufficiently widespread as to constitute a generally recognized and acceptable standard. The entire affair, therefore, represents a difficult and lengthy undertaking.

The sound ideas on the subject of pile foundations, mentioned briefly in Mr. Crandall's discussion, were presented in 1931 in a separate paper of greater length.[125] A valuable analysis of the effect of kneading on the properties of clay may be found in a paper published early in 1932 by Mr. Arthur Casagrande.[126]

[125] "Piles and Pile Foundations," by J. Stuart Crandall, M. Am. Soc. C. E., *Journal*, Boston Soc. of Civ. Engrs., April, 1932.

[126] "The Structure of Clay and Its Importance in Foundation Engineering," *Journal*, Boston Soc. of Civ. Engrs., April, 1932.

Mr. Marston's brief discussion gives an insight into the careful studies which have been conducted under his supervision. The establishment of the Metcalf and Eddy laboratory represents a praiseworthy endeavor to apply the results of scientific analysis to practical problems, and to correlate laboratory findings with field observations.

The writer is indebted to Mr. Paaswell for the definite statement that the soil studies performed in connection with the subway construction in New York brought about a substantial saving of money. This is direct evidence of the fact that funds invested in such studies may be expected to pay good dividends. It is hoped that this idea will soon be grasped by the construction world in general, so that soil research will receive financial backing comparable to that accorded investigations on other engineering materials.

Professor Griffith apparently wishes to classify the writer as a member of a definite school of thought. To this the writer objects strenuously. He refuses allegiance to any particular school, believing that such an attitude is certain to result in a narrow field of view without adequate compensation in depth of perception.

It is quite true that the writer is interested in the physicist's concept of matter, and has been guilty of discoursing at some length on grain size and shape, intrinsic stresses, and soil structure, since he does not agree with Professor Griffith's idea that "such conceptions have no place in the systematic working formulations of science." On the other hand, the writer is equally interested in the methods of theoretical and applied mechanics, involving field equations subject to singularities and boundary conditions. Perhaps there was insufficient evidence in the original paper to make this clear to Professor Griffith. As an illustration, it may be interesting to elaborate a trifle upon the theory of consolidation. If a mass of saturated clay under pressure is taken as the field, it can be shown that the hydrodynamic conditions in the field are governed by the equation, $K \nabla^2 u = \frac{\partial u}{\partial t}$, in which, K is a constant of the material, and u is the hydrostatic excess at any given point in the mass at any given time, t. The solution of this equation in conformity with a given set of boundary conditions results in a theoretical expression determining the extent of consolidation as a function of space and time. A solution for the simplest case—a layer of clay infinite in horizontal extent, drained at the upper and lower faces, and subjected to uniform stress—was published by Professor Terzaghi in 1923.[127] The writer has since generalized the theory and has investigated a number of more complicated cases. This is, without doubt, exactly the kind of attack that Professor Griffith recommends.

In brief, the writer feels that it is not only possible but highly desirable to adopt a middle course, not relying exclusively on the methods of one or the other school, but investigating the favorable features of both.

Concerning the application of Boussinesq's theory to stress analysis in soils, the writer feels that his statements have been quite consistent. He

[127] "Die Berechnung der Durchlässigkeitsziffer der Tone aus dem Verlauf der hydrodynamischen Spannungserscheinungen," *Sitzungsberichte* der Akademie der Wissenschaften in Wien, Mathem. Naturw. Klasse, Abt. 11-*a*, 1923.

merely pointed out that at best only an approximate solution is possible, but that in many cases an approximate solution comes close enough to actual conditions so that information of real value can be obtained therefrom.

Mr. Gruner's description of the field tests made under his supervision furnishes an interesting example of the manner in which properly conducted research can be used to aid in the solution of practical problems. Much still remains to be accomplished in this line; but it is encouraging to realize that the zone of uncertainty invariably associated with earthwork engineering can even now be narrowed appreciably by the use of physical measurements.

The proposed permeability apparatus shown in Fig. 44 would probably work better if it were arranged so that the water flowing through the sample were to travel downward instead of upward. An upward current tends to loosen the soil, and the results may be somewhat erratic for that reason.

The depth of the consolidation device illustrated in Fig. 46 appears to be rather large compared to the diameter. There is every reason to believe that a considerable part of the applied pressure is transferred to the walls, so that the pressure intensity at the bottom of the sample is only a fraction of that at the top. This means, of course, that the measurements will indicate a smaller compressibility than actually exists.

The proportions of the proposed device, Fig. 47, are much better. However, the writer is somewhat doubtful of the advisability of measuring the shearing resistance in the manner suggested. At first glance, it looks as if the withdrawal of the central section will produce pure shear along two horizontal surfaces, as indicated by the broken lines; but the actual situation is not so simple. The application of the horizontal force, Q, reduces the lateral stress on one side of the sample (on the left in Fig. 47) to zero, and produces an intense compression on the other side. This compression must be resisted by an equal and opposite comparison on the left side of the upper and lower elements of the sample. Consequently, the main compressive stresses within the sample will be transferred along curved paths from the right of the center element to the left of the two outer elements. This represents a rather complicated stress distribution which will not result in the desired condition of simple shear. For the present, at least, the writer believes that it is more satisfactory to use separate devices for consolidation and shearing tests.

Professor Kögler's remarks on the settlement of footings bring up the same question as that raised by Mr. Feld, and the writer's comments on Mr. Feld's discussion apply in both cases. As Professor Kögler suggests, it will be advisable, when circumstances permit, to execute a series of tests on a somewhat larger scale than usual, in order to bridge the gap that seems to exist between the behavior of model footings and the action of full-sized structures.

In conclusion, the writer wishes to express his gratitude to the discussers for the time and thought they have given to analyzing the paper and preparing their comments.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WATER PRESSURES ON DAMS DURING EARTHQUAKES

Discussion

BY MESSRS. N. MONONOBE, AND H. M. WESTERGAARD

N. MONONOBE,[34] ESQ. (by letter).[34a]—The writer has read this paper with the greatest interest, and the following comments are made in connection with it.

That the solutions obtained by Professor Westergaard have a singularity at the point, $x = 0$, $y = 0$, may be attributed to his Condition (1) that $\sigma = 0$ at $y = 0$. The writer derives a different system of solutions, in which the singularity in question does not occur, by specifying the following boundary condition:

$$\left[\frac{\partial^2 \phi}{\partial t^2} - g\frac{\partial \phi}{\partial y}\right]_{y=0} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (118)$$

in which, ϕ is the velocity potential. This condition, which is well known in hydrodynamics, stands for an incompressible fluid, but may be applied to a compressible fluid if y is small.

Let y_o denote the elevation of the wave above the undisturbed free surface, and p_o, the pressure intensity on the wall. Then, the solutions are:

$$y_o = -2\delta \sum_{r=1}^{\infty} \frac{1}{n_r \sqrt{1 - \frac{w p^2 h^2}{\kappa g n_r^2}}} \times \frac{1}{1 - \frac{g}{p^2 h} + \left(\frac{g}{p^2 h} n_r\right)^2} e^{-qr} \cos pt \quad \ldots (119a)$$

and,

$$p_o = \frac{w}{g} p^2 \sum_{r=1}^{\infty} \left(\frac{g n_r}{p^2 h} \cos \frac{n_r}{h} y - \sin \frac{n_r}{h} y\right) \frac{2\delta}{n_r^2} \times \frac{h}{1 - \frac{g}{p^2 h} + \left(\frac{g}{p^2 h} n_r\right)^2} \times \frac{1}{\sqrt{1 - \frac{w p^2 h^2}{\kappa g n_r^2}}} - \cos pt \quad \ldots\ldots\ldots\ldots\ldots (119b)$$

NOTE.—The paper by H. M. Westergaard, M. Am. Soc. C. E., was published in November, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: February, 1932, by Messrs. Theodor von Karman and Paul Bauman; April, 1932, by R. Robinson Rowe, L. Prandtl, and Cecil E. Pearce; and May, 1932, by Messrs. John H. A. Brahtz and Carl H. Heilbron, Jr., and Boris A. Bakhmeteff.

[34] Director, Research Office of Public Works, Home Dept., Tokyo, Japan.

[34a] Received by the Secretary March 4, 1932.

in which, $q_r = \frac{n_r x}{h}\sqrt{1 - \frac{w p^2 h^2}{\kappa g n_r^2}}$; $p = \frac{2\pi}{T}$; δ is the amplitude of the motion; and n_r denotes the roots of the following equation:

$$\frac{g}{p^2 h} n_r \sin n_r + \cos n_r = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (120)$$

Inspection of Equation (119*a*) will show clearly that the series converges at $x = 0$. With data as supplied by the author, $\frac{g}{p^2 h}$ is approximately equal to 1.64×10^{-2} and $\frac{\rho_0 p^2 h^2}{\kappa}$ is approximately equal to 1.8×10^{-3}. Therefore, it is seen that the answers in Equations (119) are nearly the same as those given by the author when r is small, while there is an appreciable difference when h, the height of the dam, is small. The elevation of the wave is found to be -0.47 ft.

The writer has investigated the wave motion of water contained in a rectangular tank, 17.9 ft. by 5.97 ft. by 5.97 ft. deep, produced by simple harmonic motions with periods of 0.86 and 1.68 sec. For each period the number of standing waves observed (Fig. 29) was the same as the number of

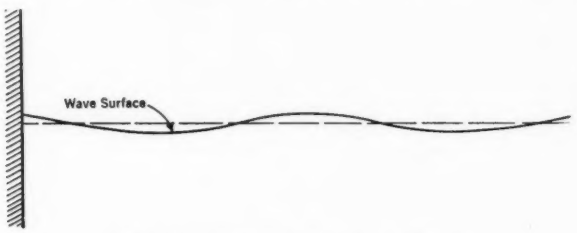

FIG. 29.—WAVES OBSERVED IN TANK.

oscillating waves. The period was nearly the same as that of the external force. For the two periods of harmonic motion, 1.68 and 0.86 sec., $1\frac{1}{2}$ waves and $4\frac{1}{2}$ waves, respectively, were observed. According to a theoretical calculation, for $1\frac{1}{2}$ waves the periods of oscillation of the water were found to be 1.59 sec. and 1.53 sec. for depths of 2.9 ft. and 4.5 ft., respectively; and for $4\frac{1}{2}$ waves the period of oscillation was found to be 0.87 sec., nearly independent of the depth.

It is not a simple matter to find a definite relation between the number of oscillations and the heights of waves. Although Fig. 30 suggests that the heights may have a tendency to increase with successive oscillations the reverse tendency was also observed.

It has been stated[35] that experiments by Professor Lydik S. Jacobsen are in fair agreement with the formulas by the author. Because Professor Jacobsen's experimental apparatus was not described in detail the writer hesitates

[35] "Earthquake Forces on Dams" by S. B. Morris, M. Am. Soc. C. E., and C. E. Pearce, Assoc. M. Am. Soc. C. E., *Bulletin*, Seismological Soc. of America, September, 1931.

to pass final judgment. Nevertheless, it seems likely that a standing wave was observed in the experiment and, consequently, he is inclined to the belief that the increase in water pressure was due to this rather than to factors involved in Professor Westergaard's theory.

In the case of a basin with finite depth, the equation of motion derived by Flamant gives the maximum total pressure, in metric units, exerted upon a unit width of a vertical wall by oscillating waves, as follows:

$$P = \frac{H^2}{2} + \frac{h_o^2}{2}\left[ln \cosh \frac{\pi H}{L} - \frac{1}{2}\tanh^2 \frac{\pi H}{L}\right] + \frac{h_o L}{\pi}\left[\frac{\cosh \frac{\pi H}{L} - 1}{\cosh \frac{\pi H}{L} \sinh \frac{\pi H}{L}}\right] \quad (121)$$

in which, $2L$ is the length and h_o is the height of the waves. With data obtained experimentally by the writer, $H = 4.53$ ft., $L = 1.97$ ft. and $h_o = 0.66$ ft., Equation (121) gives $P = 0.3199 + 0.400 = 0.3599$ tons per ft.

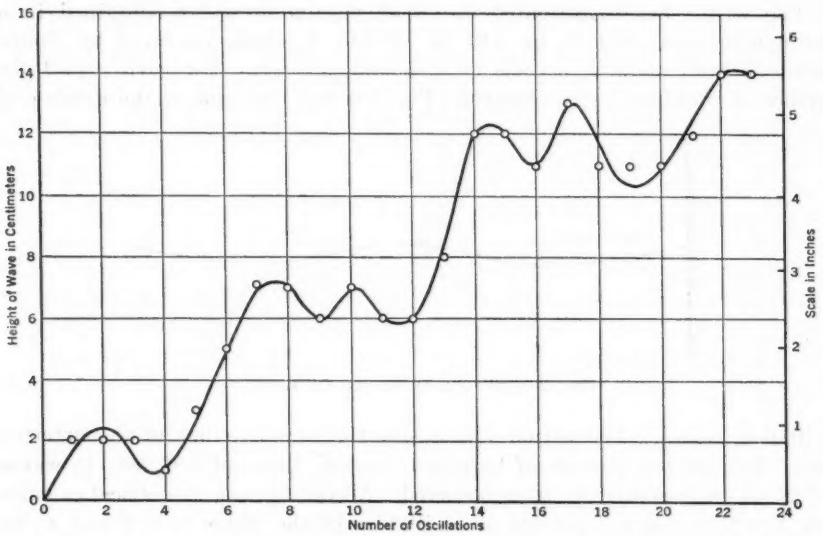

FIG. 30.—RELATION BETWEEN NUMBER OF OSCILLATIONS AND HEIGHTS OF WAVES.

If the free water surface is assumed to rise only 0.33 ft., the hydrostatic pressure would become 0.3686 tons per ft., an increase of 0.0481 tons per ft. above the original value.

It does not seem possible to obtain small enough periods of the wave-generating force by the apparatus referred to; moreover, it is evident that the results of experiments in a relatively small tank cannot be applied directly to a very large reservoir without causing considerable error. However, one does observe seiches on a lake during an earthquake and the writer believes that the systems of waves discussed here should be considered in most cases.

H. M. WESTERGAARD,[36] M. AM. SOC. C. E. (by letter).[36a]—It is gratifying that the experiments made by Professor Lydik S. Jacobsen (referred to by Mr. Pearce) showed a satisfactory agreement with the analysis. It was desirable that such tests should be made. A full account by Professor Mononobe of his tests and of his theories will be awaited with interest.

The discussions have thrown light on several different phases of the problem. Mr. Rowe, by his diagrams interpreting the theory, has made possible a clear visualization of the action of the whole body of water behind the dam. Mr. Bauman and, especially, Professor Bakhmeteff have connected the problem successfully with practical hydraulics. Dr. Brahtz and Mr. Heilbron and Professor Mononobe have obtained information concerning the effects of certain modifications of the boundary conditions. It is noted that in a majority of important cases the effects of the modifications which they studied will be small.

Particular acknowledgment is expressed to Professor von Kármán on account of his simple approximate solution. It is often more difficult to devise a simple approximate solution than to develop a complete mathematical analysis. Since the results of Professor von Kármán's theory agree so well with those of the more complex analysis, and with Professor Jacobsen's tests, the suggestion is offered that Professor von Kármán's approximate method be applied to the following problem: To determine the influence of the shape of the cross-section of the canyon at the dam when the canyon is fairly narrow.

Let, y = the elevation measured from the bottom of the water, at the dam;

B = the width of the canyon at the elevation, y, at the dam;

$A = \int_0^y B\,dy$ = the area of the cross-section of the canyon between the elevations, 0 and y;

b (as before) = the dimension of the equivalent body of water considered to move with the dam, measured from the dam in the upstream direction;

h = the maximum depth of the water.

Factors B, A, and b are functions of y only.

With the variable width, B, introduced, Professor von Kármán's Equations (55) to (59), inclusive, will be written as follows:

$$A\,a_x = B\,b\,a_y \qquad (122)$$

$$p = \rho\,b\,a_x \text{ (unchanged)} \qquad (123)$$

$$\frac{d}{dy}(p\,B\,b) = -\rho\,B\,b\,a_y \qquad (124)$$

$$\frac{d}{dy}(\rho\,B\,b^2\,a_x) = -\rho\,A\,a_x \qquad (125)$$

and,

$$B\,b^2 = \int_y^h A\,dy \qquad (126)$$

[36] Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

[36a] Received by the Secretary October 10, 1932.

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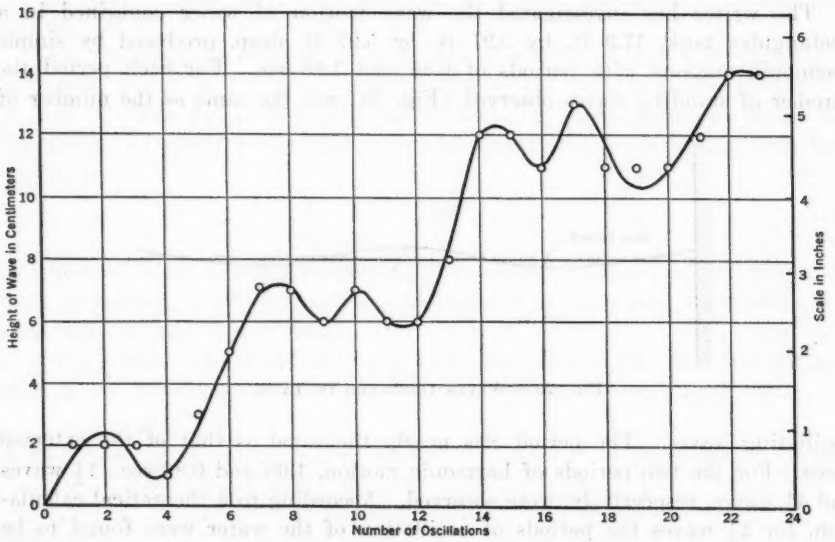

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[36] Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

[36a] Received by the Secretary October 10, 1932.

The last quantity, Equation (126), may be interpreted, if one wishes, as the bending moment at the elevation, y, in a vertical simple beam with supports at $y = \pm h$ and with a symmetrical load equal to B per unit of length at any elevation, $\pm y$.

Equation (126) gives,

$$B\,b = \sqrt{B \int_y^h A\,dy} \qquad (127)$$

from which, in any given case, the total volume of water which is considered to move with the dam may be computed as,

$$V = \int_0^h B\,b\,dy \qquad (128)$$

As an example, assume,

$$B = \frac{h}{2} + y \qquad (129)$$

This equation applies roughly to the canyon at the Hoover Dam. By Equation (126) one finds, then,

$$B\,b^2 = \frac{1}{12}\,(5\,h^3 - 3\,h\,y^2 - 2\,y^3) \qquad (130)$$

which gives, at the bottom, $b = h\sqrt{\frac{5}{6}} = 0.913h$. Professor von Kármán found $b = 0.707h$ corresponding to a constant, B. Further computations by Equations (127) to (129) give $V = 0.496h^3$, while a constant width equal to the average width, $B = h$, according to Professor von Kármán's Equation (65), gives $V = 0.555h^3$. Thus, with the variable width, the pressure at the bottom, $\alpha\ m\ b$, is greater, but the total horizontal force, $\alpha\ w\ V$, is less than the corresponding values in case of a constant width, $B = h$. By assuming values of b according to Equation (53), $b = \frac{7}{8}\sqrt{h\,y}$, with y denoting the depth of any point and h denoting the maximum depth of the water, one finds in the example of variable width, $V = 0.525h^3$, while with $B = h$, one finds $V = 0.583h^3$. It is concluded that, in the case of a narrow canyon, Equation (53), used as just indicated, leads to a value of V slightly on the side of safety.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WESTERN HIGHWAY PRACTICE WITH SPECIAL REFERENCE TO CALIFORNIA

Discussion

BY C. S. POPE, M. AM. SOC. C. E.

C. S. POPE,[10] M. AM. SOC. C. E. (by letter).[10a]—It has been a source of gratification to the writer that a paper on a practical subject of this kind should evoke such interesting discussion and cause such a concentration of experience records. A study of the comments of the various discussers will yield much of benefit to those in search of solutions of similar problems.

Mr. Bright calls attention to the wise policy of the U. S. Bureau of Public Roads in requiring that expenditures shall not be made on dead-end projects, but only on highways that are at least a part of a through traffic road.

In the West, the measures taken for the protection of the forests and the maintenance of scenic beauties are most timely and are receiving the co-operation of the various State departments.

The construction of tunnels has been a moot point, more or less, with highway engineers, many of whom prefer open cuts because of the assurance that the roadway will be always well lighted. The tunnel recently constructed under the supervision of the Bureau of Public Roads into Yosemite Valley will be studied with great interest as to the manner in which traffic is handled through it.

In Mr. Rader's opinion there seems little probability that the location of cloudbursts can be anticipated with any degree of certainty, and the writer is inclined to agree with this view, but he makes the mental reservation that studies should be inaugurated to determine, if possible, any laws that may govern the location of cloudbursts. For some years, in California at least,

NOTE.—The paper by C. S. Pope, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1932, by Messrs. J. S. Bright and R. D. Rader; March, 1932, by Messrs. H. S. Kerr, E. Q. Sullivan, and T. A. Corry; and May, 1932, by Messrs. Charles T. Leeds, and G. S. Paxson.

[10] Chf. Constr. Engr., California Div. of Highways, Sacramento, Calif.

[10a] Received by the Secretary October 10, 1932.

it was held that sand dunes could not be controlled, and that their formation and actions were erratic and could not be predicted. However, a study of the dunes in the Imperial Valley indicated that they conform to very definite laws within certain limits and that it was entirely possible to eliminate some of their disadvantages.

As to drainage structures of the extent used in some of the desert work, Mr. Rader very properly points out that there is some question as to whether such expensive drainage systems are justified by the traffic. An analysis of construction that cost about $3 000 000, in the desert section of California, indicates that nearly $1 000 000 was used in the construction of drainage channels and trestles, which, in view of a traffic running from, say, 200 to

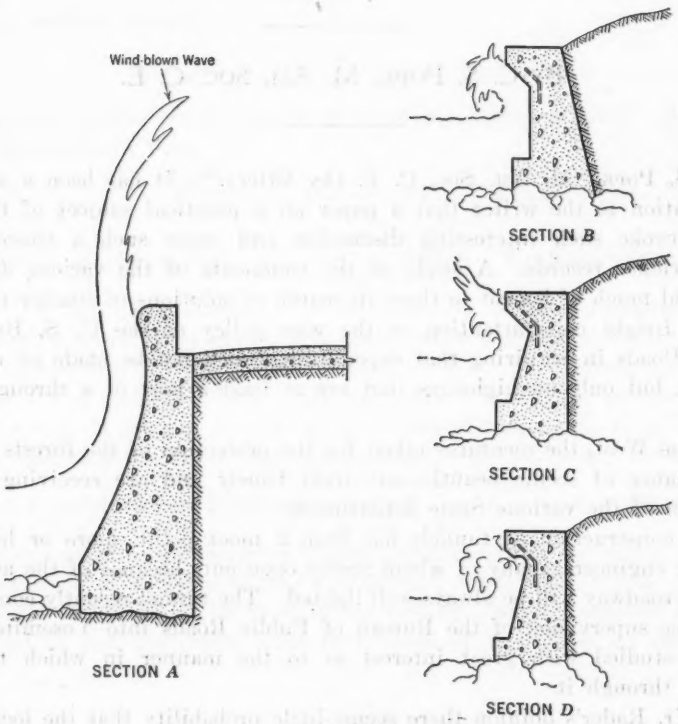

FIG. 23.

1 200 vehicles for the different routes, might give some room for argument. The drainage structures undoubtedly permit of uninterrupted traffic over these highways, whereas the construction of dips, which is the other alternative, introduces, at times, the problem of rather severe maintenance and also a less safe road. The record of five or six years in one district shows twenty-eight deaths to have occurred due to traffic accidents in dips, so that, from this point of view, there is much to be said for the more expensive construction.

FIG. 24.—COPING OF SEA WALL SHATTERED AT THIN COMPRESSION JOINT DUE TO EXPANSION OF CONCRETE UNDER MOISTURE AND TEMPERATURE.

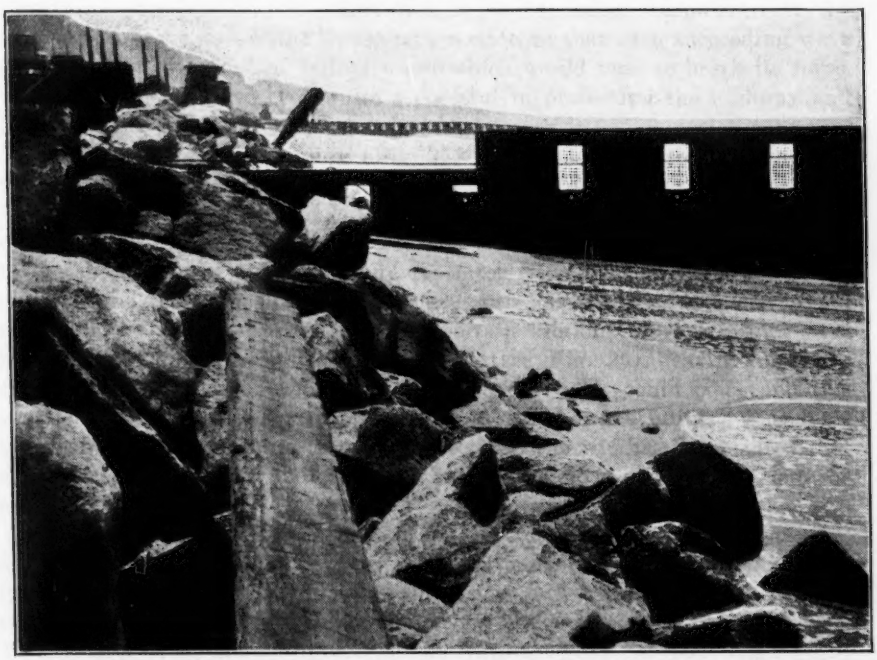

FIG. 25.—APERTURES IN PILING WALL TO PROMOTE RAPID FORMATION OF BEACH.

Colonel Leeds points out the problem brought about by moving bridge foundations in alluvial soils, such as are found in Imperial Valley, due to the movement of the entire mass of the stream bed down stream. An attempted cure for this condition was the construction of a submerged dam down stream from one of the bridges, but the method proved unsatisfactory, and it has since been the custom to drive pile trestles and replace them when serious movement occurs.

Colonel Leeds comments on the sea-wall section used by the Division of Highways on some of the work in California. The original section (Fig. 23, Section *A*), used on a wall of considerable length south of Santa Barbara, showed a throw-back section that gave no interruption to the flow of waves, and, in cases of high winds, the sheet of water was thrown over on the highway. This wall was of massive section, about $\frac{3}{4}$ mile in length, and was founded on hard shale. An unfortunate feature of design was the lack of sufficient expansion provision in the coping section of the wall which formed the throw-back, and, as a consequence, a considerable part of this coping has been shattered through pressure due to moisture and temperature (see Fig. 24). The main body of the wall, however, has shown only slight deterioration, due to the action of the sea water.

Along the rocky coast south of Section *A*, Fig. 23, a type of wall founded on heavy riprap already in place and having a vertical face (Section *B*), and a throw-back section projecting at an angle, was constructed over a considerable length of shore, but, later, the front face (Section *C*), was given a batter to break up the direct impact and scouring action of the waves. In one section, also (Section *D*), an effort was made to induce turbulence at the moment the wave struck the throw-back section so that each succeeding wave would be met by a wall of falling water which would tend to break its force. These latter walls have been quite successful in protecting the highway and in preventing spray sheets from being blown over it.

Groynes of steel sheet-piling (Fig. 25) were constructed to a considerable extent during 1931-32, in order to build up shore protection north of Santa Monica, and the technique of building these groynes has shown some advance over past practice. Each groyne was constructed at a pre-determined angle to the shore line, in order to secure the most rapid and stable accumulation of sand, and the grade line was also fixed in such a way as to increase and maintain the deposit. In some cases, certain sections of sheet-piling were made shorter than the adjoining sections (see Fig. 25), leaving openings through the sheet-piling wall through which the water could pour, and this greatly facilitated the depositing of sand in front of the piling. The durability of this type of piling has been questioned, but it has been stated that experience in Germany indicates that a satisfactory economic life may be expected.

Mr. Paxson calls attention to the use of gravel sausages in Oregon, and also to the use of Holland grass for stabilizing sand dunes. Both these methods are valuable in the proper locations. A variation of the gravel sausage enclosed in wire mesh has been developed by F. A. Kittredge, M. Am. Soc. C. E., Chief Engineer of the National Park Service. The method

requires the construction of a smaller sausage of gravel enclosed in wire mesh, which is laid beside the larger protective sausage that is subject to scour. In case of a washout, the smaller sausage sags into the depression and supports the larger structure.

In closing this subject, the writer wishes to express his appreciation of the interest evinced by the discussers and particularly of the friendly spirit in which the many criticisms were advanced.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WIND-BRACING CONNECTION EFFICIENCY

Discussion

BY MESSRS. R. A. CAUGHEY, AND U. T. BERG

R. A. CAUGHEY,[61] M. AM. SOC. C. E. (by letter).[61a]—A problem that has been generally recognized as difficult, but which has not received much attention as far as definite solutions are concerned, is attacked in a commendable manner in this paper. It will be recognized by any one who has given the problem any thought that the author's solution is not by any means the final word on the subject, but that he has made many steps in the right direction. It is not with the idea of criticism that the writer has prepared this discus-

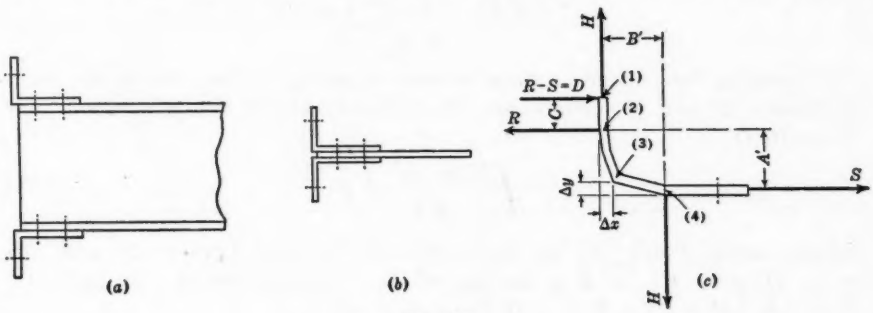

FIG. 34.

sion, but rather with the hope that some of his suggestions may help in a small way in arriving at a generally accepted solution.

Angle Connections.—Referring to the derivation of Equations (5) and (6) the writer would call attention to the fact that they have been derived in such a manner as not to call attention to the shear on the rivets in the outstanding legs of the angles, caused by the tendency of the angles to straighten. Fig. 34 shows two common types of connection. The author's

NOTE.—The paper by U. T. Berg, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: April, 1932, by Messrs. David Cushman Coyle, William R. Osgood, O. G. Julian, Harold S. Richmond, and Robins Fleming; May, 1932, by Messrs. L. E. Grinter, C. R. Young, C. H. Sandberg, N. A. Richards, Jacob Feld, Dana Young, and A. J. Wilcox; August, 1932, by Messrs. R. D. Dalton and John J. Gould; and September, 1932, by Messrs. W. C. Huntington and Paul Nielsen, and A. Toriggino and Erle L. Cope.

[61] Prof. of Structural Eng., Iowa State Coll., Ames, Iowa.

[61a] Received by the Secretary September 30, 1932.

notation is used, except that the center line of the angle is taken as the working line of a rigid frame. The shear referred to is denoted by the symbol, H, in Fig. 34(c). This diagram shows only one angle, which is one of a pair of angles such as is used in Fig. 34(a), or Fig. 34(b).

As the unit pressure on the column caused by the bending of the outstanding leg of the angle is rather small considering the material, it will be assumed that the bending is resisted by force, $R - S = D$, at a distance, C, from the rivet. Since the distance, A', does not change, it is possible to write an elastic theory equation as follows:

$$\int_{(2)}^{(4)} \frac{M x\, ds}{E I} = 0 \qquad (35)$$

Using Point (2) as an origin, it is seen that the quantity, $M x$, between Points (2) and (3) is zero, since $x = 0$. Between Points (3) and (4), $M = -D(A' + C) + R A' - H x$ and $ds = dx$. Then,

$$\int_{(3)}^{(4)} \frac{M x\, ds}{E I} = \frac{1}{E I} \int_0^B \left[-D(A' + C)\, x + R A' x - H x^2 \right] dx$$

$$= \int_0^B \left[-D(A' + C) \frac{x^2}{2} + \frac{R A' X^2}{2} - \frac{H X^3}{3} \right] \frac{1}{E I} = 0$$

from which,

$$H = \frac{3 R A' - 3 (A' + C) D}{2 B'} \qquad (36)$$

Assuming the change in angle between tangents to the curve of the angle at Points (2) and (4) equals zero, the following elastic theory equation may be written:

$$\int_{(2)}^{(4)} \frac{M\, ds}{E I} = 0 \qquad (37)$$

Again, using Point (2) as an origin, M between Points (2) and (3) $= -D(y + C) + R y$ and $ds = dy$. Between Points (3) and (4), $M = -D(A' + C) + R A' - H x$ and $ds = dx$.

Using the values of moment specified, the elastic equation is developed as follows:

$$\int_{(2)}^{(4)} \frac{M\, ds}{E I} = \frac{1}{E I} \int_0^{A'} \left[-D(y + C) + R y \right] dy$$

$$+ \frac{1}{E I} \int_0^{B'} \left[-D(A' + C) + R A' - H x \right] dx = 0 \qquad (38)$$

from which,

$$\frac{-D(A')^2}{2} - D C A' + \frac{R (A')^2}{2} - D A' B' - D C B' + R A' B' - \frac{H (B')^2}{2} = 0$$

or,

$$H = \frac{-D(A')^2 - 2 D C A' + R(A')^2 - 2 D A' B' - 2 D C B' + 2 R A' B'}{B^2} \qquad (39)$$

Equating the values of H given in Equations (36) and (39) and solving for R,

$$R = D\left[\frac{C\,B' + A'\,B' + 2\,(A')^2 + 4\,A'\,C}{2\,(A')^2 + A'\,B'}\right] \quad \ldots\ldots\ldots\ldots (40)$$

Substituting for D the equantity, $R - S$, in Equation (40) and solving for R,

$$R = S\left[\frac{C\,B' + A'\,B' + 2\,(A')^2 + 4\,A'\,C}{+ C\,B' + 4\,A'\,C}\right] \quad \ldots\ldots\ldots\ldots (41)$$

or,

$$S = \frac{R\,(4\,A'\,C + C\,B')}{[2\,(A')^2 + 4\,A'\,C + A'\,B' + C\,B']} \quad \ldots\ldots\ldots\ldots (42)$$

Having adopted a value of R, and having computed S by Equation (42), the value of H may be obtained from Equation (36), remembering that $D = R - S$.

In order to illustrate the method of procedure that the writer has in mind, Example 2 of the paper will be solved by use of the formulas proposed. The

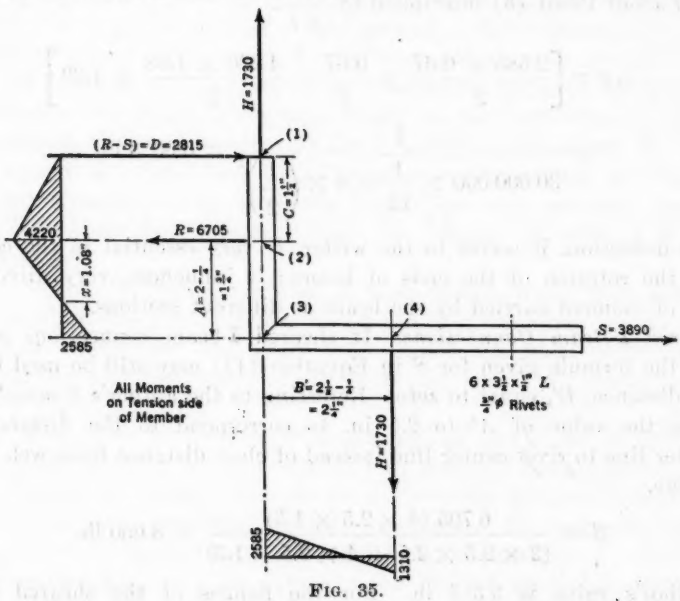

FIG. 35.

angle in question with applied forces and moment diagrams is shown in Fig. 35.

Taking into account the difference between A' and B' as defined by the writer and A and B used by the author, $A' = 2 - \frac{1}{4} = 1\frac{3}{4}$ in.; $B' = 2\frac{1}{2} - \frac{1}{4} = 2\frac{1}{4}$ in.; and $C = 1\frac{1}{2}$ in. Using $\frac{3}{4}$-in. rivets and allowing tension to the amount of one-half the initial tension as does the author, $R = \frac{13\,410}{2} = 6\,705$ lb. per rivet. By Equation (42),

$$S = \frac{6\,705\,(10.50 + 3.37)}{(6.12 + 10.50 + 3.94 + 3.37)} = 3\,890$$

Substituting proper values in Equation (36),

$$H = \frac{3 \times 6\,705 \times 1.75 - 3 \times 3.25\,(6\,705 - 3\,890)}{2 \times 2.25} = 1\,730$$

Fig. 35 shows the moment diagrams for the legs of the angle from Point (1) to Point (4). The value of S is 3 890 against a value of 3 440 computed by the author, and the maximum moment is 4 220 against a value of 3 270 by the author. The fiber stress due to bending, using 4 in. as the width of metal as did the author, is $\frac{4\,220 \times 6}{4 \times (0.5)^2} = 25\,300$ lb. per sq. in., against 19 600 lb. per sq. in. obtained by Mr. Berg. The value of X is 1.08 in. as compared to 0.95 in. obtained by the author.

The deflection, Δx, of Point (3), Fig. 34(c), may then be found from the moment diagram by the moment-area method, remembering that the deflection of Point (3) with respect to Point (2) equals the moment of the moment-area diagram about Point (3) multiplied by $\frac{1}{EI}$,

$$\Delta x = \left[\frac{2\,585 \times 0.67}{2} \times \frac{0.67}{2} - \frac{4\,220 \times 1.08}{2} \times 1.39\right] \times \frac{1}{30\,000\,000 \times \frac{1}{12} \times 4 \times \left(\frac{1}{2}\right)^3} = 0.0023 \text{ in.}$$

This deflection, it seems to the writer, is very essential in the computation of the rotation of the ends of beams; it influences, very directly, the amount of moment carried by the beam at different sections.

Sheared **I***-Beam Connections.*—If sheared **I**-beam connections are considered, the formula given for S in Equation (42) may still be used by making the distance, B', equal to zero. Referring to the author's Example 1 and changing the value of A' to 2.5 in. to correspond to the distance from web center line to rivet center line instead of clear distance from web to rivet center line,

$$S = \frac{6\,705\,(4 \times 2.5 \times 1.5)}{(2 \times 2.5 \times 2.5 + 4 \times 2.5 \times 1.5)} = 3\,650 \text{ lb.}$$

The author's value is 3 585 lb. For the flanges of the sheared **I**-beam, moments and deflection may be computed from a moment diagram as noted in the case of the outstanding legs of angles in a previous paragraph.

This method gives the results desired in a direct way and everything is taken into account that can be accounted for conveniently in a theoretical way. There may be cases in which the horizontal reaction, H (Fig. 35), on the rivets of the outstanding leg of an angle connection, with a resulting tension in the rivets of the other leg, would be very heavy. It is with the idea of taking this reaction into account and assisting in the development of a

general method for the design of this type of construction (either angle or sheared I-beam connections) that the writer submits these formulas.

It becomes evident, however, as one studies this problem that more experimental data are greatly needed. Granted that there is no appreciable deformation in rivets under initial stress and that the deflection of the connection pieces is possible of close computation, there still remains the effect of the behavior of column flanges or webs to which the beam is connected, the deflection in connection pieces due to shear, etc.

U. T. BERG,[62] ASSOC. M. AM. SOC. C. E. (by letter).[62a]—High ultimate strength for connections, as given by Messrs. Toriggino and Cope, are not surprising; nor is the shape of the deformed connection anything but natural at the later stages of loading. However, in the modern steel tower, stiffness rather than ultimate strength is the important factor. H. V. Spurr, M. Am. Soc. C. E., has shown[63] that steel tonnage may have to be sacrificed to keep deformations within desirable limits; therefore, a careful study of each connection becomes imperative.

The Wilson-Oliver tests mentioned in the paper, showing uniformly high values for the initial tension of rivets, seemed to the writer to give promise of a definite basis for the design for stiffness. With rivet stresses less than the initial tension, and, consequently, practically no rivet elongation, the joint deformation would be confined mostly to the connection to the beam. Unfortunately, the tests reported by Professor Huntington and Mr. Nielsen, do not appear to substantiate initial tension values as high as 70% of the yield-point strength. This obviously jeopardizes any practical application of the initial tension values, unless the "factor of safety" is lowered proportionally.

The test results shown on Fig. 27 undoubtedly prove that connections with more than one line of rivets have merit, but the curve for the *C* specimen shows clearly that the inner rivets reach the yield point at a load approximately only 18% greater than 43 000 lb., which is the standard working stress for $4\frac{1}{8}$-in. rivets.

That there may be inferior workmanship, as mentioned by Mr. Wilcox, does not appear to be a sound reason why rational design methods should not be attempted. Surely, the cost of the riveting work is a small part of the total cost of a building, so that time spent for inspection could be increased many times without appreciable extra expense.

The writer has used the actual area of the rivet hole rather than the nominal area, in order to conform to the test values used as a basis for the paper and given in Table 1. Of course, a hot driven rivet can not fill the hole completely after cooling. This leads directly to the question of rivet slip and its effect on the rigidity of the joint.

Several discussers have pointed out that the value, Δ_2, is incomplete and that an additional item should be included for the rivet slip. The reason for omitting it is based on the report of the Committee of the American

[62] Structural Engr., Brooklyn, N. Y.

[62a] Received by the Secretary October 24, 1932.

[63] In his book on "Wind Bracing."

Railway Engineering and Maintenance-of-Way Association[44], which concluded that:

(1) The resistance of a rivet joint against deformation by shearing forces, up to the yield point, is due to the friction between the surfaces held in contact by the rivets.

(2) The yield point of a riveted joint is reached when the shearing forces are equal to the friction of the surfaces held in contact by the rivets.

(3) The deformation of a riveted joint at the yield point is caused by the slipping on each other of the surfaces held in contact by the rivets. This slipping, is due to the "diametral" contraction of the rivets in cooling after they are driven, which leaves a space between the body of the rivets and the edge of the rivet hole.

(4) After the slip at the yield point has occurred and the rivet is brought to bear against the edge of the rivet hole, a deformation of the body of the rivet takes place with an accelerating increase in the resistance until the entire side of the rivet has been brought to bear against the edge of the rivet hole. This deformation continues beyond this point with a diminishing increase in the resistance until the ultimate strength of the rivet in shear has been reached.

An examination of the effect of a tensile force, S, acting at each end of a connection, as shown in Fig. 36, suggests that the shearing stress can not be distributed evenly among the rivets. The upper plate between Rivets R_4 and

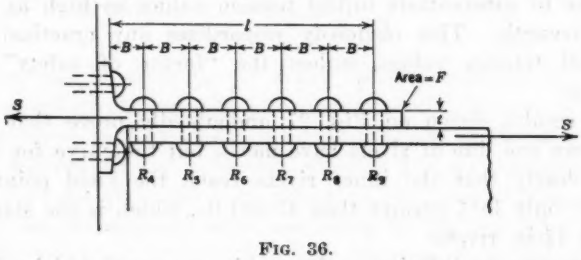

FIG. 36.

R_5 would be stressed five times as much as the lower plate with corresponding elongations. A gradual movement must take place to adjust these differences in deformations. The tests mentioned previously indicate that the friction at the plates is overcome by a force equivalent to a shearing stress on the rivet equal to, say, 10 000 lb. per sq. in., after which a gradual slipping takes place.

The force, S (Fig. 36), would first have to overcome the friction at the rivets, R_4 and R_9. A slip would then take place, but this is limited to the amount of elongation between the rivets, R_4 and R_5 and, also, R_8 and R_9. The friction at Rivets R_5 and R_8 has not yet been overcome, and the elongations are fully elastic in spite of the gradual slipping at the first line of rivets. Subsequently, the friction at Rivets R_5 and R_8 is overcome, permitting Rivets R_6 and R_7 to come into action. It is not until the friction at all rivets has

[44] *Proceedings*, Am. Ry. Eng. and Maintenance-of-Way Assoc., Vol. 5, 1905.

been overcome that any slipping not included in the elastic elongations will enter into the value of the total elongation.

The approximate value of the combined elongation of both plates can be expressed as follows:

Effect from Rivets R_4 and R_9 only:

$$\frac{S}{3}\frac{1}{E}\left(\frac{2B}{F}+\frac{5B}{2F}\right)=\frac{S}{3}\frac{B}{EF}\ (4.5)\ldots\ldots\ldots\ldots(43)$$

Effect from Rivets R_5 and R_8 only:

$$\frac{S}{3}\frac{1}{E}\left(\frac{4B}{F}+\frac{3B}{2F}\right)=\frac{S}{3}\frac{B}{EF}\ (5.5)\ldots\ldots\ldots\ldots(44)$$

Effect from Rivets R_6 and R_7 only:

$$\frac{S}{3}\frac{1}{E}\left(\frac{6B}{F}+\frac{B}{2F}\right)=\frac{S}{3}\frac{B}{EF}\ (6.5)\ldots\ldots\ldots\ldots(45)$$

The sum of these elongations is:

$$5.5\,\frac{SB}{EF}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(46)$$

Equation (46) applies to both plates. However, in a practical case, the lower plate would be the flange of the I-beam and this elongation is included in Equation (17), except that the approximation is made that the moment, M_G, is constant. Actually, the value of M_G increases from zero at the face of the support, to the full value at the last line of rivets (R_9 in Fig. 36). The angular rotation corresponding to this new form for the negative moment can be expressed as:

$$\phi=\frac{L\,M_G}{EI}\left(\frac{1}{2}-\frac{l}{2L}\right)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(47)$$

This actually increases the value of M_G compared with the straight-line assumption.

Assuming that the elongation of the upper plate is the same as that of the lower one (actually this condition may vary; on some structures the plate may be the stiffest and, on others, the I-beam flange), and keeping in mind that the lower plate, corresponding to the flange of the I-beam, is included in the deformation as expressed in Equation (17), the net additional elongation to be included in Equation (11) would be one-half the total expressed by Equation (46), namely, $2.75\,\frac{SB}{EF}$.

This would provide for values up to a force equivalent to a shear on all rivets of about 10 000 lb. per sq. in. An examination of the aforementioned elongations[64], suggests to the writer that if the full length of the connection is included in the elongation ($6\,B$ for a condition, as shown in Fig. 36), full provision is made for a slip up to an average shearing stress on all rivets equal to, say, 18 000 lb. per sq. in. Applying the full length does not materially alter the fact that as far as determining the amount of rigidity of the connection, the reduction in the fixing moment due to the connection is small and had better be neglected.

The writer believes that if the value for Δ_2 is based on an effective length of the connection equal to the full length (to the center of the last rivet), full provision would be made for any deformation that may come into play within the practical limit of the connection (up to the point when the yield point of the tension rivet is reached), and that no additional value should be included for slip of rivet or elongation of any part of the I-beam, as these conditions are all included.

(The values, $E\,\Delta_2$, in Table 3, are based on an effective length assumed to be equal to one-half the distance from the first to the last rivet plus 3 in.)

Mr. Fleming points out the importance of shear deformations. Unquestionably, they would increase the values, $\Delta_1 E$, as given in Table 3—in some cases considerably—but the influence in reducing the fixation value of the joint is not appreciable. The effect on the rivet elongation is also unimportant. The writer has computed many variations of the general Equation (4), but in all these cases the effect on the rivet elongation was small. However, the influence is important on cases similar to that given by Mr. Coyle.

In his example (in which, $2R = 2S = 2 \times 19\,160$ lb., $A = 1.6$ in., $t = 1.4$ in., $C = 1.5$ in., $I = 0.69$ in^4., and $b = 3$ in.), the bending effect is: $\frac{dy}{dx} = -\frac{WA^2}{2EI} = 0.00118 \times 1.5 = 0.00178$ in. The shear effect is expressed by, $\frac{dy}{dx} = -\frac{3W}{btE} = 0.00045 \times 1.5 = 0.00068$ in., and the combined effect is, $-0.00178 + (-0.00068) = -0.00246$ in. When the load is removed the tension is reduced an amount equal to $\frac{0.00246 \times 30\,000\,000 \times 0.518}{2} = 19\,200$ lb. Therefore, the permanent initial stress is equal to $19\,160 - 19\,200 = -40$ lb., which indicates a loose rivet.

For a connection such as that of Fig. 3(*a*), with a flange thickness varying from $u = 1$ in. to $m = 1.4$ in., the corresponding values are, as follows: The bending effect is approximately 33% more than for $t = 1.4$ in., that is, $0.00178 \times 1.33 = 0.00237$ in. The shear effect is approximately 40% more than for $t = 1.4$ in., or, $0.00068 \times 1.4 = 0.00095$ in. The combined effect is $0.00237 + 0.00095 = 0.00332$ in. In this case, when the load is removed the tension is reduced an amount equal to $\frac{0.00332}{0.00246} \times 19\,200 = 25\,800$ lb. Therefore, the permanent initial stress is $19\,160 - 25\,800 = -6\,640$ lb., which indicates the presence of a loose rivet even on a stiffer connection.

Mr. Dalton questions the assumption relative to the location of the force, $R - S$. The effective value of C is influenced, of course, by the plate thickness, hence the value, $C = \frac{1}{2}$ rivet head $+ 1\frac{1}{2}\,u$, was adopted as the maximum when computing Table 3.

Within this limit, and at the point at which the initial tension is reached on the rivet, the distance, $\frac{2}{3}\,C$, does not seem unreasonable if one is willing

to assume that the initial tension presses the plates together approximately uniformly at the stage when no outside force is acting. However, for exceptional cases, such as Test Specimen *B*, Table 7, in which the value of *C* is approximately 0.75 in., and that of *A* approximately 2.3 in., the pressing down of the edges would increase the force under which initial rivet tension is reached.

The shortening due to plate pressure is approximately $\frac{R_i t}{2 C b E}$. This leads to the following equation for the rotation at rivets:

$$\frac{R_i t}{2 C b E} \times \frac{1}{2 C} = \frac{S A^2}{2 E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

in which, $I = \frac{b t^3}{12}$. If this rotation is doubled, an approximately correct allowance would be made for the shear influence. Add this value to the original Equation (2*b*) and,

$$2 S = 2 R_i \left[\frac{t I}{A^2 b C^2} + \frac{2 C}{3 \left(X + \frac{2}{3} C \right)} \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

This makes allowance for exceptional small values of *C* in relation to *A*.

It is correct, as stated by Mr. Dalton, that the beam rotation is not included in Equations (5) and (6). At Rivet R_4, the dip in relation to the corner would be, $\Delta_1 \frac{B}{D}$, and the revised Equation (5) would be:

$$M_c = \frac{S A^2 (2 A + 3 B)}{3 A^2 + 1.5 B D + 6 A D} \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

The revised value for M_c will be found to vary in most cases very slightly, and certainly not sufficiently to justify the more elaborate form.

Equation (5) is based on the design of a beam, continuous over three supports: The first fixed at R_1; the second support at the corner; and the third at R_4. The second and third supports are at the same elevation and the first support is below this line a distance, Δ_1, but this value does not influence the equations, except in deriving Equation (50). The unknown moments are: M_c, which is constant from the first to the second supports (otherwise, the shear in this span would not be *S*); and M_4, the fixed-end moment at R_4. Then, by slope deflection and the theory of area-moments (applying the force, *S*, as a triangular moment load varying in value from *S A* at R_1 to zero at the second support):

$$S \frac{A^2}{2} - M_c A - E I \phi = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (51)$$

and,

$$- M_c \frac{B}{2} \frac{B}{3} - M_4 \frac{B}{2} \frac{2}{3} B = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

from which, $M_4 = - \frac{M_c}{2}$, and,

$$M_c \frac{B}{2} \frac{2}{3} B + M_4 \frac{B}{2} \frac{B}{3} - E I \phi B = 0 \quad \ldots\ldots (53)$$

Professor Caughey's solution should give the same result as the continuous-beam method. Equations (6) and (2*b*) give all the information supplied by Equations (39) to (42). The value, H, introduced by Professor Caughey, is the shear on the second span of the continuous beam. Its value is:

$$H = \frac{M_c - M_4}{B} = \frac{M_c - \left(-\frac{M_c}{2}\right)}{B} = \frac{3}{2}\frac{M_c}{B} \quad \ldots\ldots\ldots\ldots (54)$$

This force does not seem of much importance. It is not a shear on any rivet, but pressure from the corner of the angle to the beam. Another force, equal, but in the opposite direction, produces a tension on Rivet R_4.

The writer prefers Equation (6) because it depends on the shape of the section only. It should be kept in mind that the values, X and Y, must be solved before the true value of S can be determined. There is absolutely no discrepancy between Professor Caughey's formulas and those of the writer, if identical values for A, B, and C are used. However, the force, R-S, should not be applied on an arm, C, unless a stage beyond the initial tension value of the rivet is investigated, but then Equation (37) no longer holds true, because the elastic line ceases to be fixed at the rivet, R_1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ECONOMIC PROPORTIONS AND WEIGHTS OF MODERN HIGHWAY CANTILEVER BRIDGES

Discussion

BY MESSRS. HAROLD E. WESSMAN, HERSCHEL H. ALLEN AND WILSON T. BALLARD, AND J. A. L. WADDELL

HAROLD E. WESSMAN,[10] ASSOC. M. AM. SOC. C. E. (by letter).[10a]—The author is to be congratulated for presenting to the Engineering Profession another extensive paper dealing with the economics of an important type of highway bridge structure. In past years, Dr. Waddell has given much study to the economics of various kinds of railroad bridges, as a result of which many valuable data are now available. The intensive building of long-span highway bridges during the last decade has brought questions of their economics to the fore. In view of Dr. Waddell's previous researches, no one is better fitted to present authoritative information on the economics of spans of this type.

One of the important points brought out is that previous investigators found values entirely too high for the economic length of suspended span. When differences of opinion exist, it is necessary to examine the mode of procedure leading to certain conclusions. Maximum moments and shears due to dead load and live load, erection stresses, wind stresses, minimum sections, number and kinds of steels used in the same structure, and width of roadway, must all be considered in a series of designs in order to determine exactly the least total cost of metal possible for a single crossing. In other words, exact and complete detailed designs and weight estimates are necessary in order to draw rigid conclusions as to the most economic length of suspended span. This, of course, involves an immense amount of labor. The author used an approximate method in first making his deductions; but he

NOTE.—The paper by J. A. L. Waddell, M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1932, by Messrs. Henry C. Tammen, Robert W. Abbett, Frank W. Skinner, Charles M. Spofford, and R. Wayne Lincoln.

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[10a] Received by the Secretary September 13, 1932.

established his point by having one of his computers make complete designs and weight estimates, using the 1 050-ft. Cooper River Bridge as a basis. A series of designs were made in which the length of suspended span was varied. The effect of varying the roadway widths was also ascertained. The results demonstrated conclusively that the economic ratio of suspended-span length to main-span length was less than 0.50 for that particular crossing.

In the making of economic studies, it must be remembered, however, that variations in certain factors over which the engineer has little control, such as the difference between estimated unit prices and contractor's unit prices, the ratio of cost of alloy steels to carbon steel for a structure using two or more kinds of metal, the method of erection, etc., may affect conclusions considerably. Consequently, rigid laws of economic proportion cannot be established. That, however, does not detract one whit from the worth of an investigation such as this. The general limits for the length of suspended span, 45 to 50%, established in this paper, are of great value in obtaining an economic layout for any highway cantilever bridge.

Aesthetic requirements often tend to conflict with true economy in a bridge structure. In the case of the cantilever bridge, however, æsthetics and economy "walk hand in hand." Better outlines are obtained by keeping the length of the suspended span to less than 50% of the length of the main span. The writer has made several scaled layouts of a cantilever bridge with a 900-ft. main span, varying the lengths of the suspended span from 40 to 55% of the length of the main span. In each case, the economic proportions for the suspended span accorded with Dr. Waddell's practice for simple truss spans.[11] It was found that the lower limits for the lengths of suspended span

TABLE 9.—Relation Between Main Span and Suspended Span

Bridge	Main span, in feet	Suspended span, in feet	Percentages
New Orleans Bridge, over the Mississippi River	1 760	714	41
Longview Bridge, over the Columbia River	1 200	440	37
Carquinez Straits Bridge	1 100	433	39
Cooper River Bridge	1 050	438	42
Wabash Railway Bridge, over the Monongahela River	820	360	44
Ohio River Bridge, at Beaver, Pa	760	285	37
Outerbridge Crossing, over Arthur Kill	750	300	40
Sewickley Bridge, over Ohio River	750	350	47

gave the best structure from an artistic standpoint. With a suspended span, 420 ft. long (47% of the main span length), very pleasing outlines were obtained. In fact, there was little to choose between this and a layout with a suspended span, 360 ft. long (40% of the main span length).

It is interesting to note that most of the important cantilever bridges, proposed or built, have relatively short suspended spans. Some of these bridges are listed in Table 9.

As a by-product of his major investigation, Dr. Waddell found that it was economical to use silicon steel in simple truss highway spans as short as 200

[11] "Economics of Bridgework," by J. A. L. Waddell, Chapter XX.

ft., when the bridge carried four lanes of traffic When only two lanes were carried, the economic limit was 300 ft. These limits depend, of course, on the ratio of market prices of silicon steel and carbon steel, and the percentage of carbon steel in the silicon truss spans. The significance of this part of the author's investigation lies not, however, in the actual figures given as limits, but in emphasizing the fact that it is worth while to investigate alloy steel designs for short-span structures. Many designers have the idea that an alloy steel is something to be used only in a long-span bridge. This mistaken idea may be due in part to the fact that many technical schools teach steel design in terms of ordinary carbon structural steel, without emphasizing the economic possibilities of high-strength steels. At any rate, it is an expensive idea to cherish. There are cases in which it might even be economical to use alloy steels in girder spans.

HERSCHEL H. ALLEN[12] and WILSON T. BALLARD,[13] MEMBERS, AM. SOC. C. E. (by letter).[13a]—The author, in his paper, has provided the Engineering Profession with a contribution of great value. The writers have long felt the need of such a study in order that all the essential basic facts pertaining to cantilever bridges might be crystallized into a form readily available for use in fixing the economic proportions and quickly estimating the weights of such structures.

The writers concur in the statement by Mr. Hardesty (under "Present Study," in the main paper) to the effect that the economic proportions that governed the design of early single-track railroad cantilevers do not apply to modern highway cantilever structures.

The practice of the writers' office confirms the author's first and second conclusions under "Résumé of Results." The third conclusion states without qualification that the economic length for anchor arms of his Type *A* cantilever bridges, when the main span length is fixed, is as short as æsthetic consideration will permit. Such a conclusion does not take into account the economy of the entire structure. The topography of the site must be taken into consideration, as illustrated in the case of the alternate designs for the Ashland Bridge with long and short anchor arms hereinafter described, wherein, by lengthening the anchor arms and eliminating uplift on the anchor piers, it was possible to more than offset the additional superstructure cost by a reduction in pier costs.

Table 10 shows the dimensions of cantilever bridges and Table 11 shows in detail the distribution of weights of materials in nine highway cantilever bridges. The Huntington, W. Va.; Bellaire-Benwood, Ohio; Ashland, Ky., with long anchor arms; and the Pt. Pleasant-Henderson, W. Va., bridges have been constructed and are in service. Designs and stress sheets have been prepared for the other five bridges listed, and detailed estimates of weights have been made, but to date (1932) these latter five bridges have not been constructed.

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[12] Vice-Pres., The J. E. Greiner Co., Baltimore, Md.
[13] Vice-Pres., The J. E. Greiner Co., Baltimore, Md.
[13a] Received by the Secretary October 1, 1932.

All the designs, except those for the long and short anchor arms at Ashland, and the Norfolk, Va., design, were based on the use of medium carbon steel, with a basic working stress of 18 kips per sq. in. The Ashland Bridge, as built, contains 49% of silicon steel with a basic working stress of 24 kips

TABLE 10.—DIMENSIONS OF CANTILEVER BRIDGES AND SPECIFICATION DATA

Description	Huntington, W. Va.	Bellaire, Ohio	Point Pleasant, W. Va.	Ashland, Ky. (long anchors)	Ashland, Ky. (short anchors)	Portsmouth, Ohio	Cabin Creek, W. Va.	Norfolk, Va.
River crossed	Ohio	Ohio	Kanawha	Ohio	Ohio	Ohio	Kanawha	Elizabeth
Lengths of Bridge, in Feet:								
Main span, *L*	700	700	600	739	739	700	450	1198.5
Side spans	275	275	219.33	431	251	275	200 148.5	423
Suspended span, *l*	350	350	300	350	350	350	200	493.5
Ratio, *L*: *l*	0.5	0.5	0.5	0.473	0.473	0.5	0.444	0.412
Specification	*	*	†	‡	*	*	*	*
Live Load:								
Roadway (truck loading)	*H*-15[§]	*H*-15[§]	*H*-15	*H*-15	*H*-15	*H*-15	*H*-15	.....
Sidewalk loading, in pounds per square foot	500[¶]	500[¶]	100	100	100	100	200	.....
Roadway:								
Width, in feet	22	22	20	22	22	22	20	.....
Thickness of deck, in inches	6 to 6.5[a]	4 to 5[a]	7[b]	7[b]	7[b]	6 to 6.5[a]	7.5[b]	.....
Thickness of wearing surface, in inches	1.5[c]	2[c]	0	1.5c	1.5[c]	2.5[d]	1[c]	.....
Width of sidewalk, in feet	8	6	4	5	5	6.5	4	.....
Thickness of sidewalk, in inches	4[e]	2[f]	4[e]	4[e]	4[e]	2[f]	3[e]	.....
Medium Structural Steel (Kips per Square Inch):								
Ultimate strength	60 to 70	60 to 70	60 to 70	62 to 72	.....	60 to 70	60 to 70	.....
Working stress	18	18	18	18	.....	18	18	.....
Silicon Steel (Kips per Square Inch):								
Ultimate strength	.....	.....	.....	80 to 95	.....	.....	.....	.....
Working stress	.....	.....	.....	24	.....	.....	.....	.....
Heat-Treated Eye-Bars:								
Working stress, in kips per square inch	.....	.....	.....	24	.....	.....	.....	.....

* Final Rept., Special Committee on Specifications for Design and Construction of Steel Highway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol LXXXVII (1924), p. 1273. † Specifications, State Road Comm. of West Virginia, 1928. ‡ Specifications, Kentucky State Highway Dept., 1926. § Two 50-ton trolley cars in tandem on each track, abreast, preceded, and followed by *H*-15 trucks. ¶ Pounds per linear foot. [a] Laminated redwood. [b] Reinforced concrete. [c] Black top. [d] Asphalt. [e] Reinforced concrete slab. [f] Plank.

per sq. in., and 51% of medium carbon steel with a basic working stress of 18 kips per sq. in. Included with the carbon steel are heat-treated eye-bars with a basic working stress of 24 kips per sq. in. The Norfolk design was

based on the use of silicon steel. Tables 10 and 11 have been prepared in the same manner as Tables 1 and 2 of the author's paper in order to facilitate comparison.

Fig. 18 shows the weight per square foot of roadway of the foregoing nine designs, superimposed on the curve of Fig. 13. Bridges 7, 8, and 9 (Fig. 18), are of silicon steel; all the others are carbon steel. The writers are of the opinion that the author's weights per square foot, as shown for Type *A* bridges, with main span lengths of less than 700 ft., are too light even if they were adjusted to equivalent weights of bridges of all carbon steel in order to make them fully comparable with the writers' values for such main span lengths.

The writers agree with the author's seventh conclusion, but it is not substantiated by the data presented in his paper. The discrepancy between his seventh conclusion and his supporting data may be the result of error either

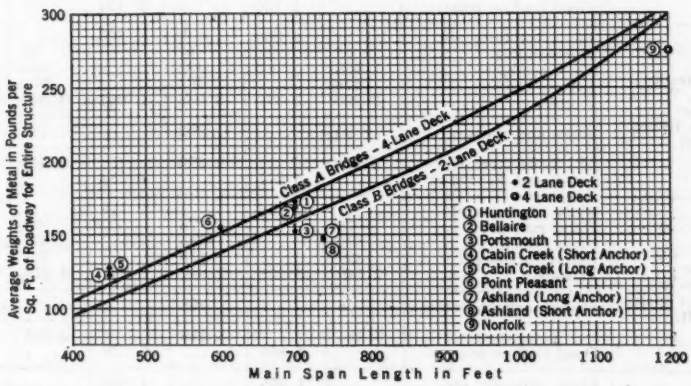

FIG. 18.—WEIGHTS OF STEEL FOR NINE MODERN HIGHWAY CANTILEVER BRIDGES.

in the weights of simple spans or in the weights of Type *A* cantilever spans as presented. It is the writers' opinion that his weights pertaining to simple spans are too high for short spans, and that the weights for cantilever bridges, as hereinbefore stated, are too low for spans of less than 700 ft., and too high for spans greater than 700 ft.

The primary features of the specifications that affect the weight of steel in seven of the nine bridges are also given in Table 10. The writers feel that such data are of vital importance in making the entire paper and discussions of practical value. It may be noted that the type of floor used on the roadway materially affects the weight of steel and that, in the cases of the Huntington and Bellaire Bridges, the live load used in the design included 50-ton trolley cars.

The value of this paper can be increased materially by the addition of similar data pertaining to cantilever bridges to be constructed in the future.

TABLE 11.—DISTRIBUTION OF WEIGHTS OF MATERIALS, IN POUNDS PER LINEAR FOOT OF SPAN, IN HIGHWAY CANTILEVER BRIDGES

Name of structure and its main-span length	Portion of structure	Anchor Arm		Cantilever Arm		Suspended Arm	
		Steel	Floor	Steel	Floor	Steel	Floor
Huntington, W. Va., over Ohio River, 700 ft.	Floor, track, etc.	.....	1 575	.....	1 575	.....	1 575
	Hand-railing	70	.....	74	.....	70	.....
	Floor system	914	.....	947	.....	860	.....
	Lateral bracing	528	.....	535	.....	303	.....
	Trusses	3 115	4 627	3 390	4 946	2 110	3 343
	Shoes and anchors	131	.....	273	.....	.....	.....
	Total steel	4 758	.....	5 219	.....	.....	.....
	Dead load on trusses	.....	6 202	.....	6 521	3 343	4 918
Bellaire, Ohio, over Ohio River, 700 ft.	Floor, conduits, etc.	.....	1 265	.....	1 265	.....	1 265
	Hand-railing	78	.....	78	.....	78	.....
	Floor system	850	.....	829	.....	780	.....
	Lateral bracing	327	.....	629	.....	262	.....
	Trusses	2 600	3 855	4 350	5 886	2 070	3 190
	Shoes and anchors	73	.....	376	.....	.....	.....
	Total steel	3 928	.....	6 262	.....	3 190	.....
	Dead load on trusses	.....	5 120	.....	7 151	.....	4 455
Pt. Pleasant-Henderson, W. Va., over Kanawha River, 600 ft.	Floor, conduits, etc.	.....	2 300	.....	2 300	.....	2 300
	Hand-railing	91	.....	87	.....	83	.....
	Floor system	520	.....	488	.....	514	.....
	Lateral bracing	476	.....	461	.....	303	.....
	Trusses	2 290	3 377	2 945	3 981	1 540	2 440
	Shoes and anchors	93	.....	292	.....	.....	.....
	Total steel	3 470	.....	4 273	.....	2 440	.....
	Dead load on trusses	.....	5 677	.....	6 281	.....	4 740
Ashland, Ky., over Ohio River (long anchor span), 739 ft.	Floor, conduits, etc.	.....	2 718	.....	2 718	.....	2 718
	Hand-railing	76	.....	76	.....	77	.....
	Floor system	532	.....	535	.....	527	.....
	Lateral bracing	390	.....	770	.....	377	.....
	Trusses	2 140	3 138	3 640	5 021	1 774	2 755
	Shoes, anchors, baffles	46	.....	298	.....	.....	.....
	Total steel	3 184	.....	5 319	.....	2 755	.....
	Dead load on trusses	.....	5 856	.....	7 739	.....	5 473
Portsmouth, Ohio, over Ohio River, 700 ft.	Floor, conduits, etc.	.....	1 255	.....	1 255	.....	1 255
	Hand-railing	75	.....	75	.....	75	.....
	Floor system	700	.....	700	.....	635	.....
	Lateral bracing	450	.....	650	.....	329	.....
	Trusses	2 450	3 675	3 490	4 915	1 816	2 855
	Shoes and anchors	73	.....	286	.....	.....	.....
	Total steel	3 748	.....	5 201	.....	2 855	.....
	Dead load on trusses	.....	4 930	.....	6 170	.....	4 110
Cabin Creek, W. Va. over Kanawha River, (long anchor span), 450 ft.	Floor, conduits, etc.	.....	2 510	.....	2 510	.....	2 510
	Hand-railing	100	.....	100	.....	100	.....
	Floor system	440	.....	440	.....	440	.....
	Lateral bracing	275	.....	440	.....	195	.....
	Trusses	1 525	2 340	2 340	3 320	1 000	1 735
	Shoes and anchors	187.5	.....	480	.....	.....	.....
	Total steel	2 527.5	.....	3 800	.....	1 735	.....
	Dead load on trusses	.....	4 850	.....	5 830	.....	4 245
Cabin Creek, W. Va., over Kanawha River, (short anchor span), 450 ft.	Floor, conduits, etc.	.....	2 510	.....	2 510	.....	2 510
	Hand-railing	100	.....	100	.....	100	.....
	Floor system	437	.....	440	.....	440	.....
	Lateral bracing	330	.....	440	.....	195	.....
	Trusses	1 636	2 503	2 340	3 320	1 000	1 735
	Shoes and anchors	287	.....	480	.....	.....	.....
	Total steel	2 790	.....	3 800	.....	1 735	.....
	Dead load on trusses	.....	5 013	.....	5 830	.....	4 245

TABLE 11.—(*Continued*)

Name of structure and its main-span length	Portion of structure	Anchor Arm		Cantilever Arm		Suspended Arm	
		Steel	Floor	Steel	Floor	Steel	Floor
Ashland, Ky., over Ohio River (short anchor span), 739 ft.	Floor, conduits, etc.	.....	2 718	.....	2 718	.....	2 718
	Hand-railing	76	.....	76	.....	76	.....
	Floor system	621	.....	519	.....	528	.....
	Lateral bracing	418	.....	665	.....	268	.....
	Trusses	2 345	3 460	2 760	4 020	1 460	2 332
	Shoes and anchors	350	.....	514	.....	.....	.....
	Total steel	3 810	.....	4 534	.....	2 332	.....
	Dead load on trusses	.....	6 178	.....	6 738	.....	5 050
Norfolk, Va., over Elizabeth River, 1 198 ft. 6 in.	Floor, conduits, etc.	.....	7 970	.....	7 970	.....	7 970
	Hand-railing	100	.....	100	.....	100	.....
	Floor system	1 912	.....	1 912	.....	1 912	.....
	Lateral bracing	1 640	.....	1 645	.....	1 175	.....
	Trusses	10 080	13 732	9 300	12 957	4 400	7 587
	Shoes and anchors	473	.....	1 420	.....	.....	.....
	Total steel	14 205	.....	14 377	.....	7 587	.....
	Dead load on trusses	.....	21 702	.....	20 927	.....	15 557

J. A. L. WADDELL,[14] M. AM. SOC. C. E. (by letter).[14a]—The writer has given due thought to the discussions of his paper and has endeavored to answer every important question raised.

While Mr. Tammen is satisfied that the approximate "short-cuts" used in preparing the paper have done no harm in the treatment of the Type *A* cantilevers, he is not so sure about their being innocuous in the case of the Type *C* cantilevers. Without these short-cuts the writer would have had neither the time nor the patience to make the investigation because he would have had to revert to his customary method of preparing complete calculations of weights of metal for four or more structures in the case of each plotted curve—a tremendous job in cantilever-bridge computation. It is true that, in some cases, the errors involved by the short-cuts were a little greater than had been anticipated; but they were not large enough to nullify the conclusions of the résumé. Moreover, the most important of them were corrected in the preliminary results, when the approximate values of the small variations from accuracy were determined; and these corrections were incorporated in the paper.

Mr. Tammen doubts whether the curves in Figs. 12, 13, and 14 correctly show the relative weights of cantilever spans and simple-truss spans. These curves were compiled partly from old records of weights that had to be adjusted to care properly for variations in loads and differences in specifications; hence, on that account, the results involve the possibility of some slight inaccuracies. Again, the personal equations of the different designers cause small divergences that are unavoidable. Their effects are partly, but never entirely, evaded by drawing a smooth curve between the plotted points instead of an irregular line joining them.

Practically nothing was known about the weights of metal for the Type *C* highway cantilevers prior to the issuance of the paper; hence, if there is any

[14] Cons. Engr., New York, N. Y.

[14a] Received by the Secretary September 20, 1932.

material variation from correctness in these three diagrams, it is likely to be in Fig. 12, which records the weights for Type *C*. That type has seldom been adopted hitherto, although known to be economic, because usually it fails to fit a crossing that calls for a cantilever layout. Such a crossing generally demands an extra long main span, because of navigation interests, or because of deep and swift water, or a dip in the bed-rock that requires a wide opening to straddle it, in order to avoid costly piers. Whenever it is feasible to use either Type *A* or Type *C* cantilevers for a proposed crossing, it will be more economical to adopt the latter, provided the length of the central span can be made about 36% of the total length of the triple-opening structure. The designer can begin with the weights of metal given in the diagram, using them for determining his dead loads, then compute his stresses, sections, and final weights of metal. It is highly improbable that any recomputing will have to be done because of serious variation between the assumed and the final weights.

The slight dropping of the cantilever curves for comparatively short spans, shown by Mr. Tammen, is curious but unimportant, because it would not be good engineering practice to adopt cantilever construction in preference to a succession of simple-truss spans of 400 ft., since the latter layout is the more rigid, and, therefore, preferable. The longer the spans the smaller is the variation in rigidity, and, consequently, the less objectionable on that account is the cantilever construction.

If in any case the probable weight of metal in the cantilever layout is not much less than that in the simple-truss layout, it is better to adopt the latter for the following reasons:

1.—The superior rigidity of the simple-truss spans, as already mentioned.

2.—The lower pound price, due not only to greater duplication, but also to simpler details.

3.—The cost per pound for erection is often greater for cantilevers than for simple-truss spans, but not always. The reason is that cantilevers are erected in a piecemeal manner, while simple-truss spans are not, unless it is found necessary to use either the method of semi-cantilevering or that of protrusion. The steadily increasing shortage of timber in this country, with the corresponding augmentation of its unit cost, has a tendency to reduce the use of wooden falsework, and, therefore, to increase the use of cantilevering, semi-cantilevering, and protrusion. For high-level bridges, especially over deep and navigable streams, the writer, of late years, has found timber falsework to be far too uneconomical; hence, he has had to adopt the use of widely spaced steel bents carried on temporary pile piers, and to cantilever between them.

The writer's practice has indicated that the average span length at which it usually pays to change from simple-truss to cantilever construction is between 400 and 500 ft. Generally, however, navigation interests call for a long main span, and sometimes also for long flanking spans, the other spans (if there are any) being of the simple-truss type, and, if feasible, of economic lengths. Other things being equal, or nearly so, the writer has made a practice of giving preference to simple-truss spans over cantilevers, and has not

countenanced, for solely economic motives, the building of the latter with main openings for Type *A* less than 600 ft. This replies to Mr. Tammen's anticipated economy for Type *A* cantilever bridges having main spans shorter than that length.

As to the effect of cost of substructures on the comparative economics of simple-truss and cantilever layouts, each case will have to be solved as an independent problem, involving generally some close detailing. As a rule, however, it will be found that this consideration will not be of paramount importance because, although the sum of the total loads on all pier tops is greater for the cantilever than for the simple-truss layout, the latter requires two small pedestals per truss, whereas the former has only a single large one per truss over each main pier. If the pier foundations are piles, the cantilever construction will be at a disadvantage; but, otherwise, the contrary condition generally holds.

In respect to the economic proportionate length of anchor spans of Type *C* cantilever bridges, the writer would state that he made two sets of calculations by different methods (although both were only approximate), and found in each case the percentage of the total length of the three-span structures to be almost exactly 36. This is a matter that hitherto has received little consideration in technical print.

Four layouts illustrating Type *C* cantilever bridges have been discussed by the writer elsewhere,[15] namely, two over the Mississippi River, at Memphis, Tenn., one over the same river at Thebes, Ill., and one over the Danube, at Cernavoda, Rumania. In each case the ratios of anchor-span length to total length vary from 0.333 to 0.34, with an average of 0.335. These ratios were probably selected for economic reasons. As these layouts are for railway structures, it is evident that the economic ratio for highway bridges should be somewhat greater, because the reason the anchor span is heavy is that it has to care for reversing stresses. These stresses are much less for modern highway bridges with their weighty floors and small live loads than for railway bridges with their light, open floors and their large live loads. For this reason it is logical to conclude that the 0.36 economic ratio found for highway bridges is just about right, especially in view of the fact that, for a structure carrying dead load alone, such as a water-pipe-line bridge, the economic ratio would be 0.4.

Mr. Tammen's dissertation about silicon steel *versus* carbon steel is interesting. The subject can be approached in several ways, and the deductions will be influenced greatly by the initial assumptions that are made. No conclusion reached would be permanent, because of the large fluctuations that are constantly occurring in the unit prices (manufactured) of the two steels, and on account of the fact that the unit prices do not necessarily increase and decrease in the same ratio.

A solution of this economic question, based upon reasonable present-day unit prices is as follows: Consider a Class *A*, simple-truss span of 300 ft., having a 42-ft. clear roadway and two 5-ft. sidewalks, and assume the manu-

[15] "Bridge Engineering," pp. 597 to 601, inclusive.

factured steel unit prices, f. o. b. cars at shops, to be 3 cents for carbon steel and 3.5 cents for combined silicon and carbon steels—also a 0.5 cent per lb. freight rate, and a 1.5-cent cost per lb. for carbon-steel erection.

The total cost of erection for the silicon-steel structure is almost, but not quite, as great as for the carbon-steel structure, the pound price for erecting the alloy steel being about $\frac{1.5}{5}(4r+1)$, in which, r (>1) is the ratio of weights of metal. From Figs. 14 and 15, $r = \frac{142}{122} = 1.164$. The remaining computations are:

Cost of erecting alloy steel $= 0.3\ (4 \times 1.164 + 1) = 1.7$ cents.
Cost of carbon steel erected $= 3 + 0.5 + 1.5 = 5.0$ cents.
Cost of silicon steel erected $= 3.5 + 0.5 + 1.7 = 5.7$ cents.
Carbon steel, 142 lb., at 5 cents = \$7.10 per sq. ft. of floor.
Silicon steel, 121 lb., at 5.71 cents = \$6.90 per sq. ft. of floor.

Hence, the alloy steel is the cheaper. If the span length were 200 ft., $r = \frac{110}{97} = 1.134$, and,

Cost of silicon steel erected $= 3.5 + 0.5 + 1.7 = 5.7$ cents.
Carbon steel, 142 lb., at 5 cents = \$7.10 per sq. ft. of floor.
Silicon steel, 121 lb., at 5.71 cents = \$6.90 per sq. ft. of floor.

This shows that 200 ft. is the span length for equal cost under the assumptions made, agreeing exactly with the limit stated in the paper.

Although these two costs are alike, the choice should be in favor of the carbon-steel bridge, because of the slightly thicker metal that would naturally be used and the consequent longer life of structure, as far as resistance to rust is concerned.

The variation between Mr. Tammen's conclusions and those of the writer in regard to silicon steel *versus* carbon steel is due to the differences in the initial assumptions. The only way to settle the matter would be by open competition among a large number of bidders; and even then the idiosyncrasies of the competitors' computers and cost adjusters would tend to prevent a truly valid comparison.

Mr. Tammen asks what general specifications were used and what was the ratio used between allowable carbon and silicon unit stresses. The answer is "those of 'Bridge Engineering' as far as they would apply, and $\frac{16\,000}{24\,000}$."

Mr. Tammen is quite right when he says:

"* * * while such economic studies as guides to conclusions are desirable tools and are often of considerable use to the engineer who devises them and has an understanding of their limitations, they must be applied by others with caution. Formulas cannot be devised to take the place of ripened experience and judgment."

Nevertheless, information of the kind furnished in this paper can be of great use to both young and old engineers who are studying or practicing in bridge work.

It is truly gratifying to note that Mr. Skinner in his discussion recognizes the importance and potential usefulness of such investigations. An endorsement by an engineer of his ripe age, wide experience, and high standing, ought to be convincing, even to the most "hard-boiled Doubting Thomas."

Both Mr. Abbett and Professor Spofford have suggested a comparison of the economics of continuous-truss and cantilever bridges. A dozen years ago the writer, in collaboration with H. Malcolm Priest, M. Am. Soc. C. E., made an exhaustive investigation of "Comparative Economics of Continuous and Non-Continuous Trusses."[16] The summary of conclusions reads as follows:

First.—For long spans the divided-triangular trussing is decidedly superior to the Petit trussing for bridges with continuous-truss spans, but not much so, if at all, for those of simple-truss spans.

Second.—For long spans there is an important saving of metal by the adoption of continuous trusses, and this saving is nearly twice as great for standard highway bridges as for modern, double-track railway bridges.

Third.—For long-span bridges the method of treating the matter of stress reversal has practically no effect upon the comparative economics of continuous and non-continuous trusses.

Fourth.—For comparatively short-span, steam-railway bridges, the continuous truss has a small advantage over the simple truss only when the divided-triangular trussing is used and stress reversals are ignored. In all other cases the comparison is either a stand-off or in favor of the simple truss.

Fifth.—For comparatively short-span, steam-railway bridges, the divided-triangular trussing is generally more economic of metal than the Pratt trussing.

Sixth.—In no case should either the Pratt or the Petit truss be used for continuous spans, because in these the divided-triangular truss is more economic.

Seventh.—In case of a structure requiring erection by semi-cantilevering, the continuous truss will possess an advantage over the non-continuous one, at least to the extent of the weight of extra metal required by the toggles for connecting temporarily over the center pier the inner hips of the two simple-truss spans. This advantage may even extend to the reinforcement for erection stresses, but if it does, the gain thus involved can never be very great.

About the only worth-while economic comparison that can be made between continuous-truss and cantilever spans is by the adoption for the latter of Type *C*, in both of which all spans are either of the same length, or nearly so. When the effect of reversion is duly considered (as it always ought to be), continuous trusses and non-continuous ones in highway bridges weigh alike for spans of 387 ft., but for spans of 775 ft., the ratio is 1.22 in favor of

[16] Presented as a memoir to the Engineers Soc. of Western Pennsylvania, and, later, incorporated as Chapter XI of the writer's treatise, "Economics of Bridgework," pub. in 1921.

continuity.[17] Assume that for intermediate span lengths the ratios can be interpolated directly. Figs. 12 and 14 of the paper and those varying ratios furnish the information given in Table 12.

It is evident that, in comparison with a layout of equal-length, continuous-truss spans of less than 500 ft., the Type *C* cantilever structure is somewhat more economic, but for greater lengths, the reverse is the case, the advantage of continuity augmenting very slowly with the span length. However, as

TABLE 12.—COMPARATIVE WEIGHTS OF METAL

Total lengths of structure, in feet	Simple-span lengths, in feet	WEIGHTS OF METAL, IN POUNDS PER SQUARE FOOT OF FLOOR, FOUR-LANE STRUCTURES		
		Type *C* cantilever	Simple truss	Continuous truss
1 200	400	138	150	150
1 400	467	159	170	166
1 600	533	190	197	188
1 800	600	210	225	208
2 000	667	236	260	233
2 200	733	262	295	258
2 400	800	292	340	287

simple-truss spans of 450 ft., for the sake of rigidity, would probably be adopted instead of cantilever construction, and as the percentage of variation for greater span lengths up to 800 ft. is quite small, it is fair to conclude that in regard to the comparative economics of Type *C* cantilevers and continuous-truss structures, there is practically no choice between them.

Mr. Abbett is right in his statement that "the economics of the arch is so vitally affected by local foundation conditions that it does not lend itself to a *carte blanche* comparison with other types." It might be pointed out that the height of roadway above water level still further complicates the problem.

Replying to Mr. Lincoln's question as to whether the economic functions determined for the paper will apply to deck structures as well as to through bridges, the answer is in the affirmative, except in so far as a small height of deck above water level may prevent the adoption of economic span lengths or of economic truss depths.

Mr. Wessman's remarks are directly to the point; and the writer takes no exception to any of them. Whether the ratio of length of suspended span to that of main opening is 0.45 or 0.5 makes very little difference in the total weight of metal, but it may in the æsthetics of the layout. It is conceivable that a case might arise wherein a greater ratio than 0.5 would be advisable, in order to meet some unusual condition, but this is not likely to occur very often.

The average of the eight actual cases quoted by Mr. Wessman is 0.41, while that of the four theoretical investigators mentioned in the paper is 0.615. Averaging these two averages gives 0.51, while, as therein stated, the writer's average of four results was 0.49; hence it may be concluded that the findings of his investigation of this economic function agree quite closely with a compromise between theory and past practice.

[17] "Economics of Bridgework," p. 80.

Answering the comments of Messrs. Allen and Ballard, the writer deems the data they have presented to be of great value to the Engineering Profession, as well as a fine supplement to his paper. For all the trouble they have taken in its preparation he tenders them special thanks.

As for the disagreement between their weights of metal and his, that is easily accounted for by the variation in specifications, especially in the items of live load and impact, which would tend to make the two plotted lines of weights cross. Furthermore, the weight per square foot of the flooring adopted would have a powerful influence in causing differences in weights of metal.

There can be no mistake of any importance in the writer's weights for Type *A* cantilever bridges, because they were based largely on actual constructions. A valid explanation of the alleged lightness in the weights of the writer's cantilever bridges with main span openings of less than 700 ft., is that for many years both he and his associates have studied deeply into the matter of economy in bridge designing, and have applied their findings in all their work.

The writer's statement in his third conclusion that "the economic length for anchor arms [of his Type *A* cantilever bridges], when the distance between main-span piers is fixed, is as short as æsthetic considerations will permit," should have had this limitation—"provided that trestle approaches are economically feasible." In the recapitulation of "the useful findings of the entire investigation," given at the end of this closing discussion, this restriction is indicated, and the possible economics of lengthening the anchor arms is discussed.

There is another economic function that has developed as an afterthought, but which should have been included in the original paper; that is, the length of the anchor arm in a Type *A* cantilever giving zero reaction on the anchor pier when the maximum live load is on the main span. This condition occurs when one cantilever arm and the suspended span are fully loaded, with no live load on the anchor arm. The live load per linear foot under this condition is that given in Fig. 1, plus the impact allowance indicated in Fig. 3, for a length of $C + S$.

The dead load for an anchor arm of any assumed length can be obtained by proportion from the dead loads of the cantilever and anchor arms already computed for the case under consideration; and no error of any account is made by assuming that both these dead loads are uniformly distributed.

With these assumptions, using the data in Table 2, and assuming various lengths in each case for the anchor arm, the results were as follows:

Name of structure.	Ratio of lengths of anchor arm to total main opening for zero reaction.
Perth Amboy Bridge	0.50
Elizabeth Bridge	0.50
Cairo Bridge	0.50
Charleston Bridge, for 1 050-ft. span	0.507
Charleston Bridge, for 640-ft. span	0.50

The slightly larger ratio for the 1 050-ft. span of the Charleston Bridge is possibly due to the great length of the main span; but the difference is so slight that it will be proper to conclude that, for zero reaction at the anchor pier, the length of the anchor arm will be exactly one-half that of the main opening in all cantilever bridges of Type *A*.

In making these computations, the effect of wind pressure was ignored; but with the almost impossible distribution of live loading that causes the zero uplift, it would be unreasonable to assume any wind pressure acting. Its effect would be to increase very slightly the 50% ratio found.

With a comparatively long anchor arm, as in the case of the Perth Amboy Bridge, an expensive pendulum support at the anchorage is not necessary. As a matter of precaution under such conditions, a slight holding-down detail can be used with a roller-bearing; or, if there is a flanking span, its end can be supported on the rolling pedestal. If the flanking span is quite long, the length of the anchor arm can be made much less than one-half that of the main opening without the necessity for adopting a pendulum support.

Before closing this résumé of the discussions, it will be well to recapitulate the useful findings of the entire investigation. They are, as follows:

Type A Cantilevers.—(A Suspended Span, Two Cantilever Arms, and Two Anchor Arms).—

First.—The economic ratio of length of suspended span to length of main opening may generally be taken at 0.5, or a little shorter if the appearance of the structure would be improved thereby. The layout involving the smallest weight of metal is not always the cheapest, because sometimes the erection conditions affect the economic length of the suspended span.

Second.—The economic truss (or tower) depth over main piers varies from 12 to 15% of the distance between centers of those piers, with an average of 13.5 per cent.

Third.—The economic length for the anchor arms, when the distance between main piers is fixed, is as short as æsthetic considerations will permit, say, 25 or 30% of the main opening, provided the depths of water beyond the anchor arms are not too great for steel-trestle construction. Should the water conditions necessitate truss flanking spans instead of the trestle work, it might prove economical to lengthen the anchor arms in lieu of using simple-truss flanking spans. This would undoubtedly effect a reduction per linear foot in the quantities of materials for piers, but might increase the weight of metal per linear foot in the anchor arms. Using the data recorded in the paper, and with due recognition of the law of variation of truss weights when augmenting the length for the anchor arms, it is a question requiring for solution a very few minutes of a competent computer's time.

Fourth.—When the distance between anchor piers is fixed, with no restriction as to the locations of the two main piers, the economic length for each anchor arm is about one-quarter of the total length of the structure.

Fifth.—The length of anchor arm giving a zero reaction on the anchor pier when the spans of the main opening are subjected to their greatest live load, and when the effect of wind pressure is ignored, is exactly one-half the length of the main opening.

Type C Cantilevers.—(An Anchor Span, Two Cantilever Arms, and Two Suspended Spans).—

Sixth.—In Type *C* cantilever bridges, the economic length of the anchor span is almost exactly 36% of the total length of the three-span layout; and even a small deviation from this economic proportion involves a material excess in the total weight of truss metal.

Seventh.—The economic truss depth at mid-length of any anchor span of economic proportions is about three-tenths that over the piers; but such a depth would be too small for appearance, and probably also for rigidity. The inferior limit should be about four-tenths the depth over piers.

General.—

Eighth.—In highway bridges it is not economical to change to cantilever construction from three simple-truss spans of equal length, unless that length is in excess of 400 ft.

Ninth.—Concerning the comparative economics of cantilever and continuous-truss bridges, there is little difference. It has been found that, in comparison with a layout of equal-length, continuous-truss spans less than 500 ft. long, the Type *C* cantilever is somewhat more economical; but that for greater span lengths the reverse is the case, the advantage of continuity augmenting very slowly with the span length.

Tenth.—In the trusses of highway cantilever bridges, the weight of metal generally divides itself between the chords and the web in the ratio of 3 to 2; and this division applies to the different spans as well as to the structure as a whole.

Eleventh.—At ordinary unit prices for metals erected in simple-truss spans, it begins to be economical of material to use silicon steel at a span length of about 200 ft. for four-lane bridges and 300 ft. for two-lane bridges; but for truly warranted cantilever structures, silicon steel will invariably be found to be less expensive than carbon steel. For spans slightly greater than these limits, especially in light structures involving thin sections of metal, it is good practice to adhere to the use of carbon steel, so as to make better provision against deterioration by rust. If the variation in unit prices, erected, for carbon steel and silicon steel is reduced in the future, as ought to be the case, the economic superior limits herein stated for carbon-steel construction in simple-truss spans will be increased.

In conclusion, the writer desires to thank most heartily the members who have discussed his paper and to express his deep appreciation of their courtesy. He had hoped for a much wider discussion, such as has been given during the past half century to most of the papers that he has presented to the Society, but he recognizes the restricted character of the present subject, and the fact that there is really not much for any one to say about it unless it be in the line of either disagreement on details or a general approval.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

APPLICATION OF DURATION CURVES TO HYDRO-ELECTRIC STUDIES

Discussion

BY C. MAXWELL STANLEY, JUN. AM. SOC. C. E.

C. MAXWELL STANLEY,[9] JUN. AM. SOC. C. E. (by letter).[9a]—Any method that appears to present opportunities for reducing the time-consuming computations involved in hydrographic studies is of interest and is worthy of careful consideration. The computations necessary in studying combinations of proposed or existing hydro-electric plants in order to determine the most feasible and economic arrangement are indeed laborious, because of the numerous variables entering into the problem, and the several solutions which suggest themselves and which must be investigated carefully.

The authors present a method of analyzing such problems by the use of duration curves. There is no doubt but that their method is a time-saver as compared to more ordinary methods involving analysis by weeks or days. That the accuracy of their method is within allowable limits for such studies is proved by the data presented on comparative results of duration curve and hydrograph methods; that is, it is proved at least for the particular conditions existing in their case. The duration-curve method presents the analysis in a compact graphical form which is visualized readily. The duration curve is not a new device to hydraulic engineers; its use in hydro-electric studies of a single plant, either run-of-river or storage, is generally accepted. The writer has used a combined duration curve for a study involving two run-of-river plants and a storage project. The authors, however, have carried the method beyond any previous stage of development; their presentation is both interesting and valuable.

The application of such a method will be as varied as the problems to which it will be applied. Its value in any particular case, however, depends upon the accuracy of the basic assumptions and the effect of the neglected factors. These items are worthy of further inspection.

NOTE.—The paper by G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E., was published in May, 1932, *Proceedings*, Part 1. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. Paul F. Kruse, Donald H. Mattern, James E. Stewart, Don Johnstone, L. T. Guy, and Joel D. Justin.

[9] Young & Stanley, Inc., Engrs., Muscatine, Iowa.

[9a] Received by the Secretary September 14, 1932.

The three basic assumptions laid down by the authors are, that the: (1) Streams under consideration have the same distribution of flow; (2) water discharged from the upper plant is immediately available at the next lower plant; and, (3) storage available at each reservoir is unlimited.

These assumptions are never true. The first appears to be the most vital to the practical application of the method, and, at the same time, the least likely of attainment. Obviously, the method cannot be applied satisfactorily without first ascertaining that the regime of the streams to be studied is sufficiently similar to avoid undue error. The failure to meet this first basic assumption will narrow the usefulness of the method. The second assumption should not generally limit the method. If the operation methods of the plants are co-ordinated in a proper manner and a reasonable amount of pondage is available, the variations can be cared for. If the plants are far apart the flow will tend to be dampened out naturally. If they are close together the assumption is practically a fact. The third assumption is not vital because, should the computed storage required exceed that available, it means simply that the problem must be re-worked on a basis of smaller outputs that will require less storage.

Therefore, in summing up the practical importance of the assumptions, it becomes evident that the first should be emphasized strongly and considered carefully. The streams under consideration must have similar distribution of flow.

What as to the factors omitted, some of which might easily be incorporated to widen the usefulness of the method? The authors list the following items: Load factor; seasonal variation of load; regulation of flow by diversion dams; variation in level due to reservoir draw-down; and evaporation from reservoir. To these might be added the following: Variation in head, due to back-water.

As stated in the paper the load factor is of extreme importance. In certain cases, the method presented may be altered slightly to permit an easier handling of the load factor. The authors use kilowatt units as ordinates. A simple conversion allows the ordinates to be plotted in terms of kilowatt-hours per day. Then, with either load factor or plant capacity known or assumed, the other is readily computed. Often, the load factor of the plants under consideration will be varied from time to time to meet changing conditions of load and stream flow. The plotting of ordinates in kilowatt-hours per day would be helpful in this respect. The growing tendency to develop hydro-electric plants as peak-load projects with relatively large capacities operating at low load factors places additional emphasis upon kilowatt-hours per day rather than merely kilowatt capacity.

Whether or not seasonal variations of load must be taken into consideration will depend upon the system load conditions. In many instances the omission of this factor will give misleading results. Where the plant will operate as a base-load plant, seasonal load variations will have little effect; but as the majority of the present-day installations are being developed as peak-load plants it would seem advisable, in order to widen the applicability of the duration-curve method, to provide some means of incorporating into

the study the effect of seasonal load variations. In some instances the regime of the river studied may be sufficiently well established so that the high flows consistently fall in certain months and the low flows in others. Where this is the case a load graph, such as Fig. 10, may be plotted, replacing the horizontal load line of the authors' graphs and permitting other computations in accordance with their method.

In cases where the regime of the river is too variable to permit such a treatment and seasonal variation of load is an important item, other methods must be devised to allow the use of the duration-curve analysis.

Any effects of regulation of flow by diversion dams might be cared for by using the regulated flow in determining the values for the duration curves.

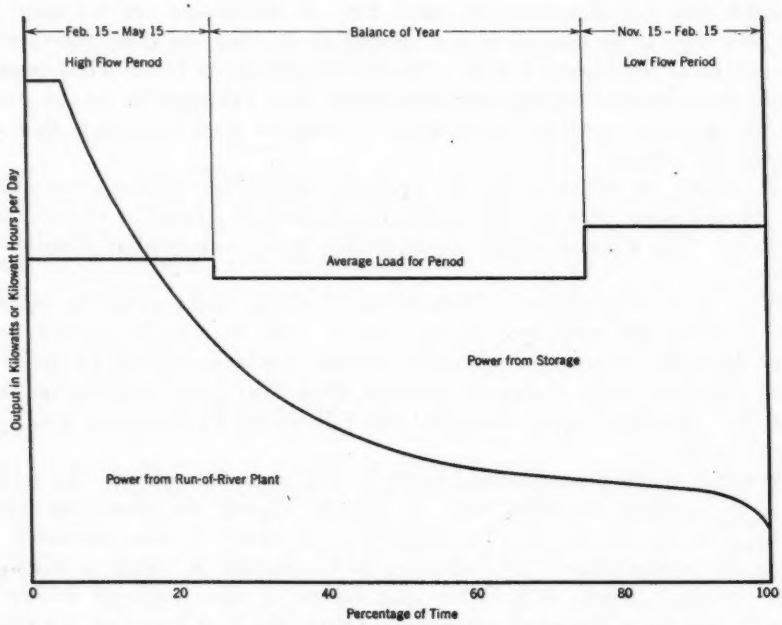

FIG. 10.—METHOD OF APPLYING SEASONAL VARIATION OF LOAD TO DURATION CURVE METHOD.

Variation of head due to reservoir draw-down will not be a factor in run-of-river plants. On storage projects, power computations using an average head should give sufficiently accurate results if the elevation-area curve for the reservoirs is a reasonably straight line through the changes of elevation required for draw-down. Such a method should certainly give more accurate results than could be obtained by omitting the loss of head due to draw-down.

A correction to the inflow of a storage reservoir can be made, which will allow approximately for evaporation. For instance, with a pond area of A acres and a total annual evaporation of d ft., an average reduction of flow

of $q = \frac{A\ d\ 43\ 560}{8\ 760 \times 3\ 600}$ cu. ft. per sec. over the year will compensate for the evaporation. In some instances, the total correction should be spread over only a part of the year.

On run-of-river plants the output is usually affected by the back-water, which reduces the head. If the power computations for the duration curve are made, using the varying heads as determined from a head-discharge curve, all error from this source may be eliminated. Equation (1) would be replaced by:

$$K\ W = \frac{Q\ e\ h}{11.8} \qquad (2)$$

in which, h varies with Q. This alteration can be made with a minimum of time if the usual data showing the head-discharge relationship are available.

Many neglected or omitted factors could have been included without materially adding to the computations involved. The inclusion of these factors should extend the usefulness of the method where it is applicable.

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DISCUSSIONS

GEORGE WASHINGTON BRIDGE: GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

Discussion

BY E. E. HOWARD, M. AM. SOC. C. E.

E. E. HOWARD,[12] M. AM. SOC. C. E. (by letter).[12a]—In his paper Mr. Ammann has produced what may be termed an epic of engineering. The simple, lucid recital outlining the controlling conditions—governmental, financial, and physical—reviewing the many complex problems and analyzing them, and including in brief compass a comprehensive story of the entire undertaking makes the paper a notable one and worthy of the great structure it describes.

The feeling, even of the non-technical observer, that the completed bridge as it looms across the great river is "right and suitable," is a tribute to the constant reversion to the fundamentals of design so evident in the recital. That a structure so monumental should be of a hybrid type is unthinkable, yet every engineer who has been involved in the complexities of problems of unprecedented structures realizes how easily one may be deflected into specious solutions apparently offering marked advantages, which in the end result in disappointment.

The simplicity of the design is most striking in comparison with the various earlier proposals. It illustrates the modern trend toward structures with the fewest number of members adequately proportioned and properly placed. Studies of possibilities always lead off into complexities, but carried to conclusion they finally reach simplicity as the ultimate and proper solution. An outstanding case is the Golden Gate Bridge, at San Francisco, Calif., which was promoted for years as a combination cantilever and suspension arrangement, but in the final analysis has become a simple suspension span, doubtless influenced by the George Washington Bridge.

NOTE.—The paper by O. H. Ammann, M. Am. Soc. C. E., was published in August, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[12] Cons. Engr. (Ash-Howard-Needles & Tammen), Kansas City, Mo.

[12a] Received by the Secretary October 6, 1932.

The history of various attempts to provide a bridge across the Hudson River is an interesting record. To earlier proponents there is the realization, not that their conceptions were at fault, but that the time was not yet ripe or the circumstances propitious. One must feel a sympathy for these pioneers of ideas.

Only with much more complete data, such as those to be published in the succeeding papers of this series, could any critical discussion of the soundness and wisdom of many determining conclusions be warranted. Nevertheless, in this paper there is evidence of such thorough and impartial consideration before reaching decisions, that any group of engineers, with all the facts before them, would substantially concur—except as such conclusions might be affected by personal preferences, or conceptions of beauty, or similar features not dependent on technical analysis.

Structures of great size are to the engineer, as the author indicates, only a question of materials and conditions and do not alter fundamentals. It is only important that size be commensurate with conditions, and it is as poor engineering to project a structure too large as too small. When mere maximum size, length, or height becomes a factor, good engineering shrinks into partial eclipse. The same care, thought, and management would be as notable if applied to a smaller structure.

The great center of population that is New York City and vicinity offers conditions widely different from any other section of the United States, or possibly of the world. The conclusion that the number of vehicles crossing the Hudson River should increase from about 5 000 000 in 1915 to 75 000 000 in 33 years seems rather incredible, but it is equally incredible that the number by actual count reached 25 500 000 by 1930—a period of 15 years. Considering the impression the vehicular traffic on the streets of New York City makes on the visitor one is inclined to wonder where 50 000 000 more vehicle on Manhattan Island are to operate.

The remarkable increases in vehicular traffic over the entire country from 1915 to 1930 were coincident with the period of multiplied production and wide diffusion of motor vehicles and the peak period of highway development. The realization of late years of the draft on the total National income of motor transportation again directs attention to the once much discussed "saturation point" in percentage of motor vehicles to population. Highway building will continue indefinitely, but doubtless at a more measured rate than for the period since the World War, which was somewhat comparable to the feverish era of railroad building which ended about a quarter of a century ago.

It would appear that other factors must enter in order to maintain the old rate of increase of vehicular movement. One such factor is the well-known and marked increase of local traffic with every mile of improved contiguous highway; but, evidently, the anticipated accelerating number of vehicle crossings of the Hudson River must arise in large measure from increasing population in the vicinity, and perhaps by encouragement of greater daily car use by additional and cheapened parking garages in New York City.

Increased crossings from long-distance travel probably cannot be anticipated as much of an element in the total traffic. Through highways for long-distance traveling are fairly well distributed over the country. Other and better roads will criss-cross counties and States and will develop and intensify local travel; but they will have relatively less effect on long-distance traffic—say, of 500-mile trips, or more.

The elaborate arrangement of approaches to the bridge is an answer as regards the immediate vicinity and an execellent illustration of proper consideration of methods of getting traffic to and away from a bridge. Public officials or, more particularly, prominent citizens promoting a bridge or viaduct which frequently will increase traffic near their property, sometimes resent cold facts of approach necessities—to be provided by appropriation of that same precious property or in other expensive ways. The writer could cite a case in which an engineer was eliminated because he recommended strongly against building a monumental structure, with a roadway 60 ft. wide, between two street ends built up with business buildings where parking, or at least stopping cars at curbs, was inevitable and where the width between curbs was only 40 ft., with no opportunity for side approaches. The suggestion that a structure only 40 ft. wide should be built and the money thus saved be used within a few years to build a parallel structure at another through street, a few blocks away, was received with disdain, evidently because it might divert traffic and build up business on the second street to the detriment of the first.

In New York City the problems are so great that the pressure of individual preference must be proportionately less than elsewhere. Experiences in smaller communities impel the melancholy conclusion that the attitude of the average good citizen toward the good of the entire community is largely measured by the Biblical axiom, "Where thy treasure is there will thy heart be also."

The excellent work of many State Highway Departments in which the Engineering Departments have relatively a free hand is accomplishing much and may be educating the public in the necessity of adequate approaches to main and restricted traffic arteries.

Of outstanding interest in design is the treatment of live load stiffening, and the author's discussion refers to early unstiffened bridges. Until a few years past there were in the Ozark Mountain country a number of so-called "home-made telephone-wire swing bridges" of spans up to 600 ft., or more, high above river gorges, which were of interest in that their crudity afforded such obvious economical efficiency. The cables were 6 in., or more, in diameter, of about 12-gauge galvanized wires simply laid together by hand, tied with occasional wire wrappings; and the hangers were frequently of loose strands of the same wires wrapped once around the cables and looped around the ends of floor-beams. The floors were a single layer of planks on wooden joists of one panel length, and provided for a single line of traffic. Although the bridges were wholly devoid of anything to function as stiffening trusses, deflections under the load of a heavy car or truck were surprisingly small, giving just enough feeling of sag to discourage speeding.

They were homely illustrations of the author's view that some arbitrary suspension bridge stiffening requirements may easily exceed practical necessities.

Another notable general feature of the Hudson River undertaking is the celerity with which it was carried forward to completion after a method of financing was finally established. The writer can recall no structure of comparative size with such variety of problems that has been brought to serviceable use so quickly. The tendency of public works enterprises to drag through controversies lagging through the Courts sometimes causes serious losses and tends to nullify the advantages of the improvement. As justice deferred may be justice denied, so public improvements begun, but unnecessarily delayed in completion, may result in detriment to a community. The writer could cite a large viaduct in a certain city which with controversies with railroads, hearings before Public Commissions, right-of-way condemnation suits, etc., is still in the Courts ten years after the community had determined the structure should be built. The detrimental effect upon adjacent property and upon other improvements can scarcely be estimated.

The accomplishment of the author and his staff and associates merits every public tribute and the entire Engineering Profession has a justified pride in this great work of fellow engineers. The people of the territory of the Port of New York Authority are fortunate that these efficient public servants do not count their work done, but are carrying forward with other great public improvements.

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DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

BY MESSRS. J. E. WILLOUGHBY, AND A. L. SONDEREGGER

J. E. WILLOUGHBY[13] M. AM. SOC. C. E. (by letter).[13a]—This paper is conclusive as evidence that forests do not conserve the water supply and that when the forest cover is replaced by small growth, the effect on flood crests is negligible. The investigations were made on small areas; nevertheless, the conclusions can be applied to larger streams and for areas differing widely in physical characteristics.

Extreme flood conditions in the streams flowing across the South Atlantic Coastal Plain are caused by tropical storms, and these storms are variable in route traversed, rapidity of movement, and quantity of rainfall. The forests, or lack of forests, are not factors of importance in the flood discharge. It is of interest to record the maximum discharge of the Savannah River at Augusta, Ga. This city is at the Falls of the Savannah and is on the western edge of the Coastal Plain. The water-shed up stream is a rolling plateau in which the streams occupy deep ravines. The plateau about the head-waters of the Savannah is broken by hills and low mountains. The virgin forests were cut long ago for lumber and the area is extensively farmed. There has been an approximately uniform increase in the area under cultivation, modified to some extent by the changed economic conditions following the Civil War between the States. The record for the Savannah River at Augusta, in extreme floods, is as follows:

Year	Maximum discharge, in cubic feet per second.	Year.	Maximum discharge, in cubic feet per second.
1796.....	360 000	1928.....	226 000
1852.....	255 000	Sept., 1929.....	374 000
1888.....	300 000	Oct., 1929.....	383 000
1908.....	307 000		

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxwell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.

[13] Chf. Engr., A. C. L. R. R., Wilmington, N. C.

[13a] Received by the Secretary August 25, 1932.

The record of this great river sustains the conclusion that the cutting away of the forests does not cause an increase in the flood discharge. In this connection, it is worth while to remember that a principal factor in the destruction of the small growth cover that follows the cutting of the virgin forests, is the policy of the forest conservationists of keeping annual fires from the cut-over forest. That policy permits the accumulation of débris on the forest floor from year to year. The accumulation soon provides fuel sufficient to produce a fire intense enough to destroy the smaller growth, and thus prevents the quick reforestation of lands unsuitable for cultivation. In the dry year, 1931, much of the young forest growth was destroyed in North Carolina, South Carolina, and Georgia, as a result of the application of the prevailing forest-conservation theories.

A. L. SONDEREGGER,[14] M. AM. SOC. C. E. (by letter).[14a]—This paper forms a timely and valuable contribution to the knowledge of the function of the forest relative to stream flow. The writer agrees with the authors in their deductions, however, having clearly in mind that these deductions apply to water-sheds with residual, and, hence, with relatively shallow earth cover, where deep penetration of rainfall cannot take place. Possibly special stress should have been laid upon this feature, because water-sheds and forest growth on formations that originate in glacial moraine, or that consist of alluvial fills, may perform a function distinctly different from that described in the paper.

The writer is familiar with Fish Canyon water-shed and its steep slopes covered with residual soils, which probably average less than 8 ft. in thickness. It is also inferred from the description of Water-Sheds *A* and *B*, at Wagon Wheel Gap, Colo., that the soil is not of sufficient depth to permit of deep penetration beyond the reach of roots. The statement[15] has been made that the augite-quartz-latite that comprises the entire foundation of the two water-sheds breaks down, by reason of its fine, crystalline structure, into a rather fine and compact clayey loam. The report states, however, that because of the steepness of most of the water-shed slopes, this quality is only partly developed; that is, steady sheet erosion prevents the accumulation of deep masses of soil and causes rock fragments to comprise a considerable proportion of the mass, sometimes as much as 50% of the first 4 ft. of soil. Furthermore, erosion and leaching tend always to rid the soil of clay and silt, leaving the coarse sand. The result on all the steeper slopes is a quite permeable and well-drained soil layer, the depth of which has never been directly investigated on the main slopes. This description, the steep slopes, and the relatively young and slender tree growth, indicate a shallow earth cover.

The intensive development that has taken place in the semi-arid West during the 20 or more years since about 1910 has brought about, in many regions, a demand for water supplies in excess of those naturally available

[14] Cons. Engr., Los Angeles, Calif.

[14a] Received by the Secretary September 19, 1932.

[15] *Monthly Weather Review, Supplement 30*, p. 3.

It is realized that the evapo-transpiration requirements of the water-shed cover may consume from 40% to 100% of the seasonal precipitation, so that the run-off available at the foot of the range may be only a fraction of the meteoric supply. It has also become apparent that flood waste is, in many cases, only a relatively small portion of the total run-off, that the cost of its conservation is economically prohibitive, and, from the standpoint of water supply, that it is of minor importance.

Salvage of even a portion of evapo-transpiration losses being more promising, there have been, therefore, many proposals for the improvement of the water-shed with a view of increasing its water yield. Broadly speaking, they may be classed in two groups[16]:

First.—"Super-forestation" to the effect of producing a denser growth of native or other vegetation on the water-shed, in order (*a*) to "keep the water in the water-shed," as the saying goes; and (*b*) to induce more copious precipitation.

Second.—"Light deforestation" to restrict the vegetation to a minimum compatible with effective prevention of erosion, the intention being to diminish the evapo-transpiration requirements of the cover.

"Super-forestation" may be described as an effort to utilize the storage capacity of the water-shed to its maximum by increasing the absorption of precipitation and preventing flood waste by means of profuse vegetation. It presents, therefore, the logical solution for water-sheds overlying deep alluvial fills or glacial moraine, in regions enjoying abundant precipitation, where water supplies are required mainly for domestic purposes and depend upon underground supplies, springs, or unregulated stream flow. Forestation of bare water-sheds would have a similar beneficial effect under such conditions.

In the Middle Western and Middle Northern States, climatic conditions and the deep alluvial deposits of the plains or the moraines of one-time glaciated areas generally favor forestation as well as super-forestation. In semi-arid regions, super-forestation may be desirable if surface storage is lacking or flood run-off is abnormal because of a meager plant cover. Under the latter condition, however, the increased consumptive use of a densely grown water-shed cover has a tendency to reduce seepage run-off because, generally, vegetation is exceptionally tolerant. Vegetation is capable of sustaining growth in wet years almost as long as the moisture supply holds out, so that, in some measure, evapo-transpiration requirements may be proportionate to the density of vegetation.

It is quite apparent that super-forestation has its natural limitations because by natural selection the density of the existing vegetation, as a rule, is the one best adapted to the prevailing climatic conditions. Attempts to "make two trees grow where formerly grew one" over large areas, therefore, would have little chance of success. On the other hand, the forestation of bare areas overlying deep earth deposits unquestionably will promote perco-

[16] Unpublished report by the writer to the Water Conservation Committee of the Irrigation Division.

lation and increase ground-water supplies and normal stream flow at the expense, however, of a lesser total run-off. This result, under certain conditions, may also be desirable in semi-arid regions.

Practical proposals of super-forestation include the planting of trees in cut-over or burnt-over areas and the substitution of chaparral by trees. As a means of retarding surface run-off and inducing seepage, a dense growth of brush is likely to be as effective as a forest. Whether or not trees would grow in the chaparral zone may depend upon local conditions.

"Light deforestation" has been proposed, with a view of reducing evapotranspiration losses of the water-shed cover, and correspondingly increasing the water crop. It may present a logical solution for water-sheds having a shallow earth cover, in which roots penetrate to underlying bed-rock, and the seeped ground-water remains exposed to abstraction by plant growth until intercepted by surface drainage. Under such conditions, a large percentage, if not all, of the seepage is consumed by plant life, reducing the seepage run-off to only a fraction of the meteoric supply. In a general way, this was the condition prevailing in the water-sheds of Wagon Wheel Gap, in Colorado, and of Fish Canyon, in Southern California.

The immediate effect of light deforestation is a greater surface run-off and lesser rainfall penetration, and the subsequent effect, a lesser evapotranspiration loss. This result may be desirable in cases where the water demand exceeds the natural run-off and where storage of flood waste is practicable either in surface reservoirs or underground in the flood-plain. It applies to a large portion of the water-sheds of the semi-arid West.

Of the many proposals to effect light deforestation, the following deserve to be mentioned: The systematic burning of the underbrush of forested areas and of the chaparral of brush-covered water-sheds; the substitution of heavy water-using species of brush or trees by others of lesser consumptive use and the cutting of trees growing along watercourses, such as willows, alders, and sycamores. It is believed that measures of this kind would contribute to a conservation of water supplies. Whether the gain would justify the risk involved, or the destruction of the charm of the shaded brook, is quite another question.

Considering the matter strictly from the standpoint of conservation of water supply, both groups—super-forestation and light deforestation—have their merits, depending upon climatic conditions and the formation and depth of subsoils.

It may be of interest to note that super-forestation (also forestation) is constantly urged and promoted in the semi-arid Western regions with a view of increasing water supplies under conditions that are inimical to the success of the plan; that is, for steep mountain water-sheds with a shallow cover of residual soils and a mean rainfall less than the optimum use of the native vegetation.

Upon closer analysis it is found that the idea of super-forestation has been brought West, as an adjunct to the great process of interstate migration which has been so active during the twenty years (1912-1932), particularly from the prairie States. Most of the population of California, for example,

settled in that State within the last generation, and having firmly in mind the benefits derived from forestation and super-forestation in their native Middle Western States, are now promoting the plan with remarkable tenacity, for conditions quite different. It is probable, for example, that the suggestion of replacing chaparral by forest originated with these people, under the claim that trees are better suited to keep the water in the water-shed and are inducive of more copious precipitation. The plan has been tried in Southern California, but failed because the trees would not grow without irrigation (except in a few favored spots).

From the water-supply standpoint, the plan has no merit: First, because the trees, like the brush, season after season, would consume the field capacity of the soil within the reach of their roots, with no chance of salvaging seepage water; and, second, because a fire (and fires do occur) would destroy the tree growth. Then, unless new trees were planted, the water-shed would remain bare until, in the course of years, a new growth of chaparral had returned it to its original condition. On the other hand, burned-over brush areas naturally recuperate within two or three years to the point at which the new growth, together with grasses and herbs, protects the soil from erosion. It is a fact that the ordinary brush fire neither destroys the roots nor burns up the seeds of herbs and grasses that seem to be buried in the soil, so that with the first rains an effective start is made toward the creation of a protective cover.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE MARTINEZ-BENICIA BRIDGE

Discussion

BY MESSRS. N. F. HELMERS, E. J. SCHNEIDER, AND M. F. CLEMENTS

N. F. HELMERS,[3] ESQ. (by letter).[3a]—One of the outstanding features in the building of the foundations of this bridge was the constant use made of triangulation in measurements and layout. It is not unusual, of course, to resort to this method of locating piers for long crossings, but in this case a detailed system of signals was developed, permitting the use of triangulation to spot every pile in the falsework for each pier.

The base line was established approximately parallel to the bridge and $\frac{1}{4}$ mile down stream from the bridge site. From the two observation stations, all locations were given not only for the falsework, but for the sinking of caissons, erection of shaft forms, setting of anchor-bolts, etc.

In connection with pile-driving, signals to the driver were made by the use of a diamond-shaped target revolving about a center. One-half the diamond was white and the other half, red. The signaled direction to the pile-driver is indicated by the white half of the diamond target, which under ordinary visibility could be seen plainly without glasses, from any pier. For driving the falsework, the pile-driver was spotted at the intersection of two lines sighted from the two observation stations, the direction having been previously calculated to form such intersection for each pile as designated on the contractor's pile plan. To obtain the necessary degree of accuracy in turning the angles, each transitman turned from his foresight on the movable disk which was set to take care of the odd minutes and seconds of the arc. The vernier was not used. This method gave the location within a probable error of 1 in. The co-operation between the instrument party and the pile-driver crew was so good that a number of times 20 piles were driven in 2 hours under these conditions. Because of the swift current it was necessary then to stop and cap the driven piles in order to prevent their being broken off by excessive and continued vibration.

NOTE.—The paper by W. H. Kirkbride, M. Am. Soc. C. E., was presented at the meeting of the Structural Division, Sacramento, Calif., April 24, 1930, and published in September, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Siems-Helmers, St. Paul, Minn.

[3a] Received by the Secretary May 9, 1930.

The main construction equipment of this work consisted of a heavy floating pile-driver, equipped with 90-ft. leads and using a 13 200-lb. double-acting hammer. The outstanding features of this machine was that, in addition to the pile-driving equipment, it carried a revolving derrick with a 75-ft. boom and had an unusual number of engine drums and winch-heads. The machine carried four 2 700-lb. anchors, each of which was handled on wire rope and operated by steam. In addition, it could carry two or more bow and breast lines; it also carried two heavy jet pumps, delivering water at 250-lb. pressure, through 2½-in. jet pipes.

Because of the need of pulling the falsework piles after the pier was completed, the machine was not rigged for batter piles, which were driven by out-hauling the hammer instead of battering the leads. It may be of interest to note that the piles in question ranged in length from 100 to 125 ft., with a 24 to 26-in. butt, and a minimum tip of 10 in.

The second important item was a 2-yd. floating concrete plant with a 104-ft. tower. It was equipped with its own "whirley" derrick for feeding the bins, as well as a bow derrick for handling spouts and tremie pipe. This machine handled tremies as long as 160 ft. for under-water concrete. Its normal capacity was 60 cu. yd. of concrete per hour. It was equipped with weighing devices to permit rapid change in aggregate proportions. It also carried extra engines to handle four anchors on wire-rope lines.

In addition to the foregoing there were two heavy floating derricks, five stiff-leg derricks erected on the falsework of each main pier, five tugs, a locomotive crane, a pump barge, a complete machine shop, and a number of deck lighters. The cement was handled in two heavy barges, housed, and provided with an inclined belt conveyor, power-driven, which delivered the cement on the platform back of the mixer on the concrete plant barge.

Practically all the machinery used was operated by steam, with oil as fuel. The tugs were operated by Diesel engines. All the plant used on this job was the property of the contractor.

Pile-Driving.—At each of the eight main piers, as soon as the first falsework piles were driven, a carefully detailed report of its behavior was wired to the contractor's consulting engineer, who estimated the depth to which the steel shell would sink and determined whether or not the mud should be removed from the inside of the shell before placing the sand filling. These studies and recommendations proved wonderfully accurate in forecasting the behavior of each caisson.

One of the important items in handling this job was the behavior of the mud on the bottom of Suisun Bay. It has been demonstrated that ordinary river silt gives sufficient bearing to be entirely satisfactory for the method used. Only a slight settlement occurred at Piers Nos. 13 to 18, where ordinary river silt was encountered.

When falsework piles were driven at Pier No. 12 a very soft bottom was found. The 100 to 110-ft. pile sank 43 ft. under the weight of the hammer and pile, and moved 5 ft. under the last ten blows. Both shell and caisson showed some movement under these conditions, and when the dredging in the caisson had gone through the sand-filling and had relieved the mud

under the cylinder of sand, both the steel shell and caisson listed so much that, at one time, the caisson was 18 in. out of plumb and the shell nearly 8 ft. At certain times the dredging would be 5 ft. below the bottom of the dredging wells on the high side (which was the maximum permitted), and as much as 12 ft. above the bottom of the cutting-edge of the low side. The caisson was landed within 6 in. of the correct theoretical position.

The softest ground of all was at Pier No. 19. The material was described on the original drawing as "blue mud with vegetation" and the falsework piles used were from 120 to 126 ft. long, extending practically to the rock. From 100 to 110 ft. of shell was used in the sand island for this pier. The weight of the sand island pushed the soft material from under it, moving a part of the falsework upward with the flow. Instead of settling uniformly at all times the shell and its sand-filling settled spasmodically, sinking 3 or 4 ft. on one side and then leveling itself in the next few hours. On April 22, 1930, this shell was 7 ft. out of level.

An attempt to cure this by loading with sand and jetting on the high side was practically unsuccessful, but by dredging for 24 hours on the high side the shell was brought to within 1 ft. of level, when the dredging was stopped. Additional sand was then added to get it still nearer level. The reason for this procedure was to obtain most, if not all, of the settlement before work on the caisson or pier proper was begun.

An interesting detail of the method used was that the maximum weight is secured at the start; that is, when the sand-fill was completed and 25 ft. of caisson added. From that point the weight of the sand removed and the water displaced by the concrete outbalanced progressively the increased weight of the concrete caission.

Under this method of sinking, the sand between the steel shell and the caisson can always be removed, thus relieving skin friction and giving the greatest weight desired to permit rapid sinking. Under normal conditions a caisson can be sunk at the rate of 10 ft. in a 4-day cycle, and, at times, this was reduced to 3 days.

E. J. SCHNEIDER,[4] M. AM. SOC. C. E. (by letter).[46]—As long ago as 1910, the writer saw designs of a proposed structure across Suisun Bay. Nothing really developed until about 1928, when the psychological moment seemed to appear, and it was apparent that the two car ferries would soon have to be replaced.

The Engineering Department of the Southern Pacific Company was ordered to make an estimate showing that it would be possible to build the bridge. For preliminary purposes, more than twelve estimates were made of steel for various combinations. Single-track, double-track, and gauntlet-track were considered, as well as high-level and low-level. The "low-level" requirement was that the under-clearance would be 20 ft. above mean higher high water, and the "high-level," 70 ft. Studies were made to determine whether a highway should be built in connection with the bridge, and esti-

[4] Contr. Mgr., Bridge and Structural Dept., Columbia Steel Co., San Francisco, Calif.
[46] Received by the Secretary May 29, 1930.

mates were prepared with and without the highway, as well as for the construction of the highway at some future time. Furthermore, carbon, heat-treated, and silicon steel were considered in connection with continuous, cantilever, and simple spans. This gives an idea of the various combinations under consideration.

The final choice was for a typical span consisting of a through Warren truss designed to carry a double-track load equivalent to *E*-90, followed and preceded by a uniform load of 7.5 kips per lin. ft. The total weight of the steel was 42 000 000 lb., of which, 25 000 000 lb. was silicon steel used for the main built-up truss members, floor-beams, and stringers; 4 000 000 lb. of heat-treated eye-bars for the bottom chord of the fixed truss spans; and the remaining 13 000 000 lb. was ordinary carbon steel.

Although it is customary to test to destruction, one or two eye-bars from every heat, the question was asked, "How do we know that the ones that are not tested are all right?" This was given much consideration because of the size of the bridge and the amount of money involved, and it was finally decided that each bar should be proof tested. As a result, the 1 807 bars were tested in the full-sized testing machine, at Ambridge, Pa., and every bar came well within the required limit. This confirms the opinion of the engineers of the fabricators who have always maintained that the eye-bar is one of the most dependable sections in a structure.

Mr. Kirkbride has explained why this particular location was chosen. The type of superstructure was selected because of economy of design, ability to resist earthquake shock, simplicity of repair, and simplicity of erection.

When the preliminary studies were made, it was first thought that a span of approximately 408 ft. would be an economical length; but the economical curve was about horizontal between 400 and 500 ft., and the question was settled when the War Department ruled that the distance between center to center of piers should be 530 ft. This decision was not reached, however, until the Army engineers had made several tests of long tows in the vicinity of the bridge where swift tides and cross-currents frequently occur. Their experiments indicated that a longer span was necessary to provide ample clearance and to prevent long tows from being "side-swiped." The War Department ruled on a 70-ft. under-clearance from mean higher high water for all except the lift span. This means that all river craft can pass under the bridge and only sea-going vessels will be required to use the Government channel where a lift span is provided with 300-ft. horizontal clearance and 125-ft. vertical clearance above mean higher high water.

The difficulties encountered in building the foundation due to the strong current and deep water and great depth of solid rock have been described in the paper. If these factors could have been avoided, the erection would have been much simplified.

All the through spans have the same panel length. The deck span, which Mr. Kirkbride has mentioned as doing so much work, has the same panel length, except that the two end panels were fore-shortened so they would go between the piers instead of resting on them. Reference was also made to the shelf or offset at the top of the caisson proper which was at Elevation

—20 below mean lower low water. This is one of the important details required in connection with the erection of this falsework span as this shelf supported a temporary steel bent on which the deck falsework span rested. The erection of the steel began from the south end and in building temporary falsework and turnouts, as much as possible of the material going into the main span was used temporarily. Two 80-ft. girders were used, which later were incorporated in the south approach; and two panels of through 506-ft. deck span were also used temporarily in erection.

The barges for floating operations were those originally designed and built for the erection of the Carquinez Highway Bridge about 7 miles down stream from the Martinez-Benicia Bridge. They were 40 by 130 ft., with 10-ft. depth, and had a capacity of 1 000 tons. When used on the Carquinez Bridge they carried a maximum load of 800 tons. When loaded to 1 000-tons capacity they had a free-board of 1.5 ft., and extra waling strips were provided on the side so that the deck would be kept dry in case of rough weather.

The first span was floated on February 28, 1930. Advantage was taken of the low tide when the barges were placed under the span. Shortly after noon, the floating was completed, and the span was resting on the temporary bents between the piers. The bents supporting this span were of sufficient length so that the span when erected on this falsework would be about 3 ft. above its proper elevation. Hydraulic jacks were used to take care of any adjustment which might be necessary.

When the lift span was swung, the falsework span was returned to the temporary dock where it originally rested and was re-assembled to the remainder of the span. As the lift span was only 300 ft. long, only eight of the twelve panels of this deck span could be used.

It was not quite so easy to land the deck span in its final position as might be expected because the north bank is not at right angles to the track. The rock-fill embankment is at an angle of about 45° and, therefore, it was not possible to warp the north barge alongside the pier as in deep water. The plan adopted was to warp the span in on a skew so that one shoe could be placed in its permanent position. This shoe rested on a turn-table center and the span revolved about this point. A falsework platform with a system of rollers was provided to accommodate the other pedestal. As the weight was transferred to this falsework platform, the inner barge was removed and the span swung into place as the outer barge was warped into its proper position.

M. F. CLEMENTS,[5] M. AM. SOC. C. E. (by letter)[5a]—In "Conclusion (2)" of his paper, Mr. Kirkbride lists the artificial "sand island" as one of the unusual features worthy of special notice.

The use of islands for the lowering of concrete or timber caissons is not new. The first use that has come to the writer's attention was a record of construction of a bridge foundation at Pittsburgh, Pa., in the Eighties.

[5] Bridge Engr., N. P. Ry., St. Paul, Minn.

[5a] Received by the Secretary September 29, 1932.

The Romans used mounds of rock or artificial islands on which bridge piers were built. They also used timber caissons, and it is reasonable to assume that they used the timber caisson and artificial islands in combination. The use of the rigid steel shell to enclose sand to form artificial islands is new and was suggested by the writer to the contractor, and his proposal to the Railroad Company for the piers at the Martinez-Benicia Bridge was submitted as a substitute for floating creosoted timber caissons.

The plans submitted to contractors in the invitation for bids were examined and the difficulties of building a floating caisson with a high tide, a fast current, and a very soft underlying material, were noted. The high tide and fast current would naturally prevent the seating of a floating caisson, as there would be a continual washing away of underlying material. This wash would then increase the area of floating caisson exposed to current; it would require more anchorage; and would make it more difficult to locate the caisson accurately. To overcome the foregoing natural conditions the writer recommended the artificial island method as being most suitable for doing the work.

It came to be known on the work as the "sand island" method. Among the advantages in its use, the writer lists a number, as follows:

(1) The element of danger that enters into the handling of large, expensive floating caissons is taken by the lighter, less expensive, preliminary shell, thereby reducing the risk in handling deep-water, deep-foundation construction work. The writer considers the safety element of the use of the "sand island" method to be the most important, and it was the principal reason for recommending its use on the Martinez-Benicia Bridge.

(2) The construction of an all concrete caisson entirely free from the use of timber permits actual contact of the concrete caisson walls and the concrete placed in dredging wells. In salt water infected by teredo or other marine borers, a wooden floating caisson must be protected by an outer layer of creosoted material, which adds to the cost of that type of construction.

(3) Concrete was poured above water where the mixture could be controlled and the deposit made in the dry. In comparison with the floating caisson, this is a distinct advantage in that a floating caisson cannot be made water-tight; therefore, tremie concrete had to be poured, and it would be impossible to obtain 3 000-lb. concrete, such as cited by Mr. Kirkbride, on account of the presence of water in the wall form of the caisson.

(4) With the "sand island" method the bottom part of the caisson is substantially supported by the sand, and high strength reinforced concrete can be made. The structural steel cutting-edge is merely a metal covering which is comparatively inexpensive; whereas, with the floating caisson, the cutting-edge is a rigid frame supporting superimposed water-tight compartments in which concrete is placed. The cutting-edge of a floating caisson is of necessity of greater weight and cost than one used on a caisson constructed on an artificial "sand island."

(5) The structure was spotted accurately over its final location at the beginning of the work.

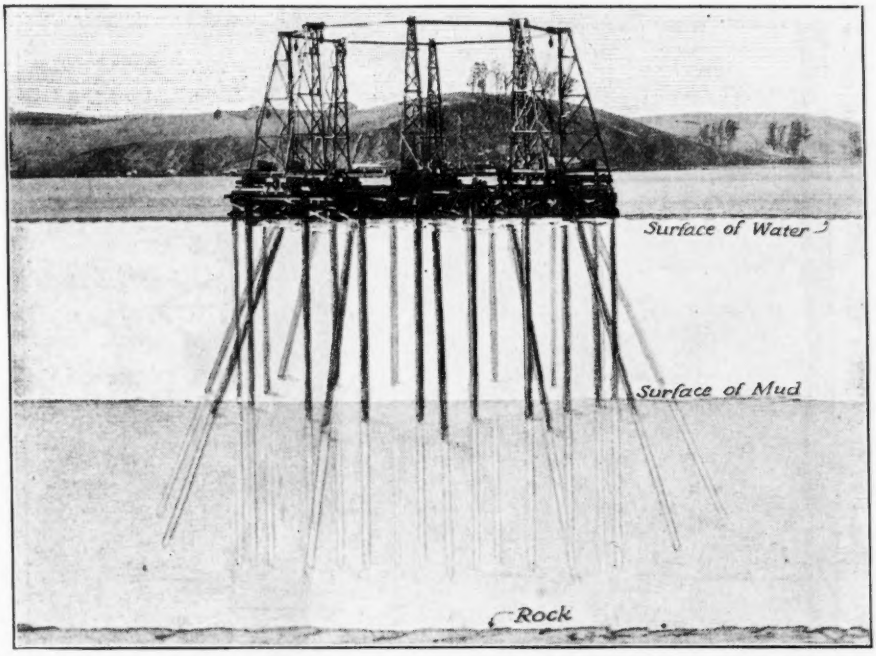

FIG. 16.—FALSEWORK FOR PLACING STEEL SHELL FOR ARTIFICIAL SAND ISLAND.

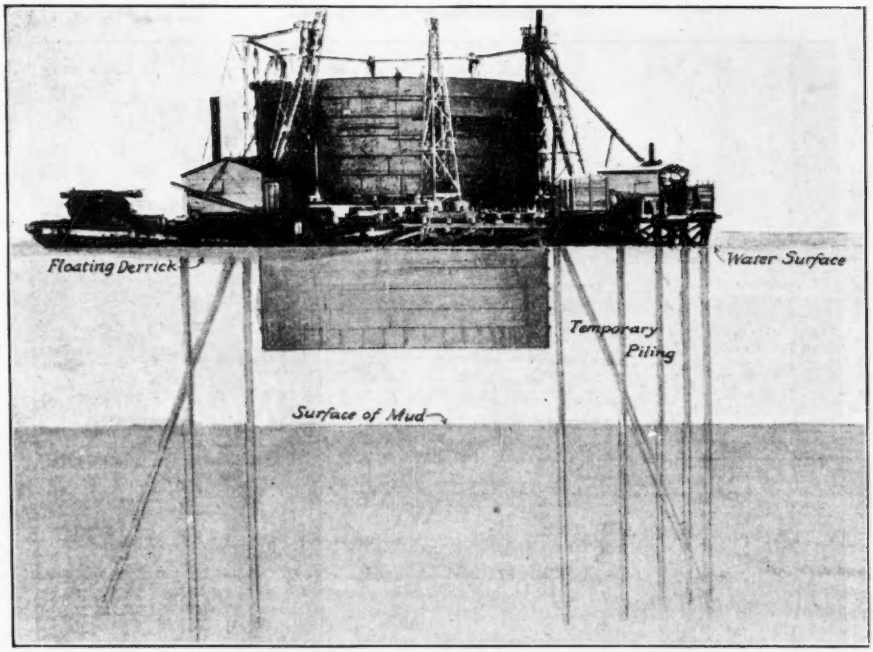

FIG. 17.—ASSEMBLY OF STEEL SHELL FOR ARTIFICIAL SAND ISLAND.

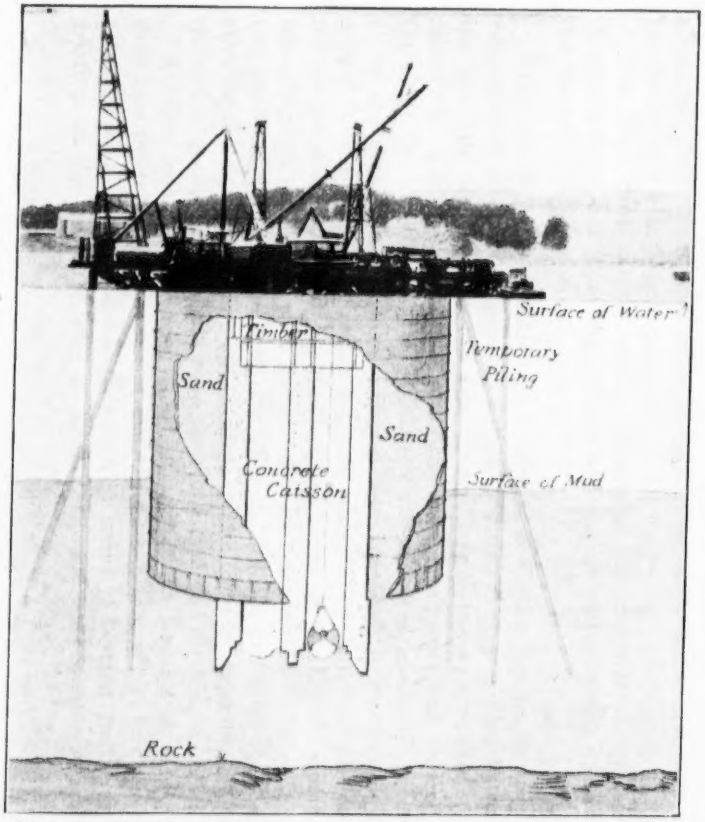

FIG. 18.—SINKING CONCRETE CAISSON THROUGH STEEL SHELL AND ARTIFICIAL SAND ISLAND.

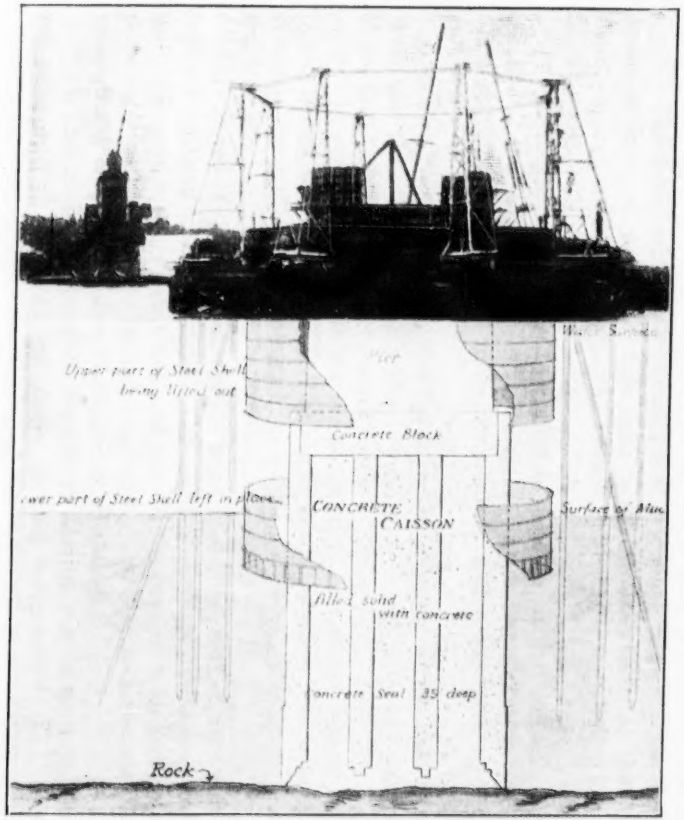

FIG. 19.—REMOVING STEEL SHELL FROM ARTIFICIAL SAND ISLAND.

(6) The caisson was controlled during the sinking process for the entire depth. The sand acting as an equalizer of frictional value for the different classes of material, penetrated below the natural ground line. At this bridge there was no sudden sinking as the caisson entered the mud below the sand in the shell.

(7) The caisson could be controlled in landing on the bottom. At the location of the bridge there was a fluctuation of tide amounting to 10 ft. and a velocity of current of 5.8 miles per hour in either direction, due to ebb and flow tide.

(8) The "islands" weigh the same as the caissons which pass through them and the action of high water, winds, waves, and tides has no greater effect than they do on the permanent piers.

(9) The method results in a marked saving in cost that varies in amount by the extent to which shells and falsework can be re-used. The actual costs resulting from the construction of "sand islands" were exceedingly close to the estimate.

(10) Speed of construction by the "sand island" method is the result of a better co-ordination of the work. The speed of sinking the caisson depends upon the speed of its construction and this can be handled with greater speed on a rigid rather than a floating structure. The Martinez-Benicia Bridge was actually completed before the anticipated date.

(11) The method reduces the amount of floating equipment required and can be applied with a minimum amount of fixed equipment.

(12) Steel sheet-piles have been used for enclosing sand in shallow water; but it is not of sufficient strength for the greater depths and had its use been attempted at the Martinez-Benicia Bridge, it probably would have resulted in the loss of two islands and caissons.

(13) The "sand island" method eliminates the necessity of constructing launching ways and providing anchors and anchor barges, which are used in the floating type of caisson.

As a matter of information, it is well to call attention to the predicted movement of islands and caissons during sinking. This was done by analyzing the data obtained on the pile-driving during the construction of the falsework. The shells were assembled on I-beam supports and lowered into the mud. As a matter of fact, the shells were lowered to an elevation 5 ft. above the mud line and then, with all lines released, it was permitted to drop 5 ft. to force a penetration into the mud. The analysis of pile-driving for the purpose of determining the depth to which the steel shells would sink into the mud, was made with surprisingly accurate results. Some difficulty was experienced at the last deep pier (No. 19, Fig. 2), and after the pier had been landed on bed-rock, it was evident, in the light of experience, that all the action of the caisson could have been predicted and all delay avoided.

Another lesson was learned from estimating the possible sinking of the shell into the mud as the result of analyzing the action of the test pile. The value of the pile to support load is both in terms of bearing on the tip and friction on the sides. A pile of known size and approximate weight was

lowered until its tip touched the mud. It was then released and the penetration from its own weight noted. A hammer weighing approximately 13 kips was placed on the pile and its further penetration noted. The bearing and friction as determined by these tests were applied to the bearing area and sides of the steel shell and the final depth to which the shell would be sunk, was determined. In most cases, the depth was predicted to within 1 ft. However, that is not so important as the fact that the prediction was made for a shell, the bearing area of which was one hundred and fifty times the bearing area of the pile.

These computations were based on ultimate values. In other words, the ultimate value of the mud or earth in bearing for the larger area was determined by the smaller area. The writer believes that tests made on soils on small areas will give fairly accurate bearing values for larger areas. This has been a disputed point, but the results of the tests of pile-driving at the Martinez-Benicia Bridge tend to prove the statement.

As a result of the experience at this bridge, it is the writer's opinion that deep piers can be placed by the "sand island" method in tidal waters under adverse conditions of high winds and waves, using circular shells 400 ft. in diameter in water 150 ft. deep.

It may appear in first analysis that the cost of all large sizes is excessive, but the safety factor outweighs all disadvantages. It can be used where the abnormal conditions, due to wind and waves, make a floating caisson dangerous.

Under the heading, "Construction," the author had described the steps involved in the artificial sand island method. Fig. 16 is a view showing piles driven to support the octagonal platform. It also shows the eight gallows frames mentioned in the paper. In Fig. 17, the steel shell is being erected and lowered. A view of the third step, showing the concrete caisson being driven through the sand island to solid rock, is given in Fig. 18 and, finally, Fig. 19 demonstrates the final step of removing the steel shell.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PRE-QUALIFICATION OF CONTRACTORS

Discussion

BY MESSRS. EDWARD W. BUSH, A. B. EDWARDS, SAMUEL T. CARPENTER, AND LEON F. PECK

EDWARD W. BUSH,[3] M. AM. SOC. C. E. (by letter).[3a]—This paper is timely and it is hoped its publication will lead to a general discussion of the subject. Pre-qualification is economic, but just now the procedure needs protection from some of its most ardent friends. Some of them are suggesting methods that are too complicated to be workable; they aim to develop certain facts of little importance; and they omit the development of other facts of prime importance on which the determination of the fitness of the contractor for the award necessarily depends. This question of the determination of the qualification of the bidder to receive the award is exactly parallel to a part of the method followed by any reputable surety company when judging the acceptability of a contract bond case; but an examination of many of the questionnaires indicates that their authors have not recognized this fact and, apparently, no attempt has been made to benefit from the very extended experience of the sureties along the same line. A surety is keenly interested in the fitness of a contractor to perform the contract properly, because if it errs in its judgment on the case it may suffer a loss. All large sureties are in touch with all kinds of construction projects in all parts of the United States and have learned by experience that human nature re-acts in about the same way on all of them. The sureties have a seasoned experience on the methods of determining the qualifications of a contractor, which if called for would assist those about to establish any scheme of pre-qualification.

When a contractor fails he is generally forced to retire from the business or continue with greatly lessened activity. Just why he failed may not be fully recognized by him, because he had never failed before, and he is apt to think that all the "breaks" were against him, etc. One never listens to a

NOTE.—The paper by C. J. Tilden, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, New York, N. Y., January 22, 1931, and published in September, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Engr., Fidelity and Surety Dept., Aetna Casualty & Surety Co., Hartford, Conn.

[3a] Received by the Secretary September 23, 1932.

technical paper read by a contractor telling why he made a mess of his business. Active and successful contractors therefore are, perhaps, not the best qualified to outline workable pre-qualification schemes because they have not been intimately associated with trouble cases as are the sureties that pay the claims. Engineers, also, are apt to lack any extended experience in the defaults of contracts. If at any locality defaults were of frequent occurrence the premium rates would be increased greatly over the present small charge for the bond protection.

The writer is not in full accord with some points of the paper. The author defines pre-qualification as "the determination of responsibility before plans and proposals are issued for bidding." Any such procedure would necessarily cause a known list of prospective bidders to be made up well in advance of the letting, and this might prevent an outsider from breaking in on any local group that possibly might have matters fixed to its entire satisfaction. The main object is not to make an award to an unqualified bidder, but this is quite removed from a scheme of pre-qualifying all the bidders before giving out the plans. A more workable way is to state in the call for bids that no bid will be received and opened from a bidder who has not qualified himself for the contract. With such a procedure most, if not all, of the prospective bidders will qualify well in advance of the letting, but all of them will know that it is possible for some other bidder to qualify himself just before the opening, and all the benefits of open competition are thus preserved. Many contractors will not know until after they review the plans and visit the site whether or not they wish to bid, but the author's definition outlines a procedure which would tend to keep this class away from the bidding.

It takes time and considerable skill to verify and evaluate properly the technical and financial information submitted by any unknown contractor, and such a check should be made by the owner before making an award to any contractor not well known to be qualified. Sufficient time can be taken after the opening for such an investigation, but to make a complete investigation of all bidders before giving out the plans would be a huge task the time for which is seldom available unless the owner is willing to receive the bids from six to eight weeks after the advertisement appears. Since adequate time is, in general, not available and since, considered as a practical matter, about all the owner can expect to accomplish before the opening is to collect enough data to enable him to form a preliminary opinion that the contractor should be allowed to bid, the owner making a complete check of the bidder's affairs only if he happens to be the low bidder. This is a practical way to work pre-qualification.

If the general information on a contractor has already been filed it is available of course for subsequent lettings; therefore, only a comparatively small amount of additional up-to-date information is needed to develop a rather full understanding of the status of the bidder at future openings. It is desirable to have the pre-qualification scheme as simple as possible both in forms and procedure so as to attract instead of repel the best bidders.

The author presents parts of a California law as "a more extended definition" of pre-qualification. These excerpts show that, at its discretion, the

Department of Public Works may call for questionnaires and financial statements before giving out the plans to bidders; and if the Department is not satisfied with the "sufficiency of the answers" it may refuse to furnish the bidder with the plans, etc. Apparently, the Department can neither follow up references nor verify the financial worth of a prospective bidder as it can only read the answers submitted. It is not charged with the duty of making an investigation other than along the narrow lines of the Act. No doubt the Department does not restrict its investigation to the narrow limits just stated, but the Act is not, in itself, a good definition of a workable scheme of pre-qualification. Further, it might be inferred from the wording that a contractor must have had a previous "experience in performing public work" in order to qualify, and this might bar a well qualified contractor if his experience had been gained wholly on work for a railroad company or a public utility. A better definition would be along the line of the following:

No bid shall be received until the bidder shall have shown before the bids are opened that he has had a comparable experience in the same or similar classes of work (if a corporation, the information shall be submitted on the experience of the personnel), also has or can obtain sufficient plant and equipment, also is possessed of sufficient financial resources as reasonably to justify the conclusion that the bidder is qualified to perform the proposed construction contract as well as continue any other contracts on hand.

In each case decisions regarding the prospective bidders shall be reached before the bids are opened, and if a prospective bidder is deemed unqualified his bid shall be returned if offered, and it shall not be received and opened.

Such a wording, if incorporated in a law or ordinance, leaves the officials a desirable freedom of action regarding the method of pre-qualification. At times an informal method could be followed. A further investigation before the award is not disbarred because all awarding bodies are now supposed to exercise discretion in the award; the new matter charges them also with the duty of investigating the bidders before the opening.

The author lists five points or elements that determine the principle of pre-qualification. He ignores other points such as the skill and ability of the contractor and the skill and experience of the contractor's organization. The surety company always collects information about these points in connection with a large contract because it wants to know that the organization will continue to function in the event that the contractor personally becomes incapacitated. Another most important element is omitted by the author as well as in the California Act, namely, information about the size, location, kind of work, and approximate percentage completed of all other contracts on hand. Are the other contracts progressing satisfactorily and profitably? A contractor has only one plant and one financial strength with which to perform all his contracts and, in the past, many new contracts have been taken by contractors with the hope that the new work will in some way supply funds needed to bolster up bad conditions already developed on other jobs. No scheme of pre-qualification is workable that neglects to give full consideration to the uncompleted work of the bidder. The sureties always investigate the other work, except in connection with small bonds written for strong contractors about whom rather full information is at hand.

The author's Point (2) indicates that the condition of the plant should be considered. It may be located here and there on other work and its condition would be impossible to determine except at a prohibitive cost. It is desirable to ask the average contractor to submit a list of the large pieces of plant, to state specifically what new plant, if any, will be needed to perform the new job if awarded to him, and to state the cost of such new equipment. The cost of new plant comes from the present working capital or net quick assets of the contractor, and, therefore, the question is always asked by a surety when underwriting a contract bond. The fact that a contractor owns plant of a certain kind is an indication that he has had experience doing the kind of work suited to the plant, and information of this kind is needed when qualifying a contractor. It is possible for a contractor to be well qualified and not own any plant, provided he knows what he should buy and has ample funds with which to purchase. The Hoover Dam was let to a new corporation having nothing but a personnel of marked ability and several million dollars in the banks.

The author presents the history of pre-qualification, its alleged advantages and disadvantages, etc., in a concise and admirable manner. The present trend among engineers and public bodies is toward a more careful selection of contractors before receiving the bids, and toward making investigations before awarding contracts. Reputable contractors favor these practices because they prevent unfair competition. The sureties, also, are in favor of the exercise of more discretion by the owners when placing contracts because no surety ever has the chance to bond an unqualified contractor until after a contract has been awarded to him. The author points out the interest the Associated General Contractors of America has exercised in helping to establish the Bureau of Contract Information, Inc.; but, apparently, in some way he missed the fact in his investigation that from its inception a select group of surety companies also has been active in the affairs of the Bureau. To date this group of sureties has contributed the major proportion of the funds used to maintain the Bureau.

The author closes his admirable paper by stating that some form of questionnaire is highly desirable, which will exhibit fully and clearly all the facts, etc., regarding a contractor. It is comparatively easy to qualify or underwrite a capable and experienced contractor as long as he keeps the scope of his operations well within the amount warranted by his working capital. When he attempts to go beyond his safe limit of volume of work more investigation of his affairs is needed and mostly this is in the line of checking carefully his alleged financial strength. The manifestly unfit are quickly disposed of, but the class between these two is the most numerous and it is also the class that causes, or should cause, the most investigation. Merely requiring answers to certain questions will not in itself establish for this class the fitness or disqualification of the bidder to receive an award although the answers will probably direct the course of the inquiry. Some of the questionnaire forms are good, but more of them are very poor working tools for an engineer or a public works body. Any of them could be improved

and as this important subject is still in a state of flux, it would seem desirable that an organized study should be made to develop the best thought regarding it. Recently, many bills on pre-qualification have been offered to State legislatures. Some of these bills have proposed non-workable schemes advocated by zealous but poorly informed individuals or societies. Many persons desire the benefits of pre-qualification, but do not know a good workable way to obtain them.

A. B. EDWARDS,[4] M. AM. SOC. C. E. (by letter)[4a]—To accept a bid without a demonstration of the aptitude, ability, integrity, and experience of the bidder would be economically foolish. In contrast with this clumsy procedure is the policy of determining the qualifications entering into the make-up of the bidder who can and will co-operate with the engineer to give the owner his money's worth.

The bidder should be able to assume full responsibility for the planning and execution of the work of construction in accordance with the commercial requirements of the owner, as outlined by the engineer. The bidder in whom confidence and responsibility is thus placed, will be developed by pre-qualification.

One potent aid in the establishment of pre-qualification as a permanent policy may be the equitable handling of candidates for membership in the fraternity of bidders. There are those who have served their apprenticeship without submitting to the enervating influences of expediency, who are adequately trained in sound procedure, and who are worthy of financial backing. Pre-qualification can and should give these canditates "elbow room" by combating the unworthy element.

SAMUEL T. CARPENTER,[5] JUN. AM. SOC. C. E. (by letter).[5a]—That some form of pre-qualification or qualification is necessary to determine the responsibility and financial standing of contractors is undeniable. To what extent it can be developed in order to place no restriction on competitive bidding is problematical. For the public's interest the presentation of a surety bond should not be taken as conclusive evidence that the finished product will be satisfactory. Specifications for public work usually carry a clause demanding the contractor to guarantee his work fully from defects for at least a year after completion. In the writer's opinion too much dependence is placed upon this clause to insure a satisfactory product. The engineer deserves a finished product that will be a tribute to his engineering skill, and the legitimate contractor deserves a protection against those who are obviously unable to complete a building program successfully, either financially or technically.

Professor Tilden states five basic criteria for determining the fitness of a contractor to receive plans and proposals for bidding. In addition, it might be well to investigate his organization, personnel, and their experience.

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[4] Asst. Engr., Div. of Met. Planning, Commonwealth of Massachusetts, Boston, Mass.

[4a] Received by the Secretary October 3, 1932.

[5] Structural Eng. Apprentice, McClintic-Marshall, Pittsburgh, Pa.

[5a] Received by the Secretary October 10, 1932.

In 1929 the State of Indiana adopted a form of qualification that provides for a questionnaire to be submitted by the contractor with his proposal. Section 1 of the Indiana statute[6] that covers this form of qualification stipulates that when any public work aggregating $5 000, or more, is to be constructed or repaired under contract, it must be let to the best of the lowest bidders. Section 2 provides means of classifying the bidders. Each bidder must submit a sworn statement of his experience, his proposed plan, and the equipment he has available for doing the work. Standard forms for this purpose (General Form 96-*a*) are supplied by the State Board of Accounts, which are designated the experience questionnaire, plan and equipment questionnaire, and the contractor's financial statement.

LEON F. PECK,[7] M. AM. SOC. C. E. (by letter).[7a]—The writer fully realizes that a brief should be submitted whereby some lowest bidders should not be awarded the contract. Evidently, this discrimination has been practiced unfairly for many years. It is not new, but those interested are trying to set up a method of making the award with fairness.

The fact that engineers are discussing means of doing this would indicate that there might be some question as to whether it can be done fairly both to the contractor and to the public, where public funds are involved.

This is a big step, seemingly in the opposite direction from that recommended in 1912 by Maj. Cassius E. Gillette, then President of the American Society of Engineering Contractors which had an active existence over a period of a few years before the World War interruption.

Major Gillete stated in a prepared paper,[8] that he had made investigations of three sets of public contracts and found a loss of $8 600 000 out of a total of $18 000 000 worth of contracts. He said it was partly due to the clause in specifications granting discretion to the awarding officials. He also stated that in all these cases, covering dozens of contracts, the invariable accompaniment of the cheat was the scarcity or absence of real competition, and that under the shelter of that potent word, "discretion," fraudulent and unwise public contracts had been let and interpreted so as to cause the loss to taxpayers of the United States of untold millions, and to rob hundreds of bidders of their just rights.

Major Gillette believed that the American Society of Engineering Contractors should sponsor a bill in every State Legislature in the United States, making it compulsory that no request for bids on public work should be advertised until the appropriation was available, and that the contract must be awarded to the lowest bidder. He had had such wide experience and had given this subject such prolonged study, that it would be unfair not to give his views cognizance at this time.

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[6] Acts, 1929, Chapter 202, p. 680.

[7] Supt., Dept. of Streets, Hartford, Conn.

[7a] Received by the Secretary, October 14, 1932.

[8] "Competition in Public Contracts", *Journal*, Am. Soc. of Eng. Contractors, Vol. 4, Pt. 1, (1912), p. 87.

Report of Tellers on Second Ballot for Official Nominees

"October 15, 1932

"To the Secretary,
American Society of Civil Engineers:

"The Tellers appointed to canvass the Second Ballot for Official Nominees report as follows:

"Total number of ballots received	3 086
"Deduct:	
Ballots from members in arrears of dues	139
Ballots not signed	9
Ballots with printed signatures	1
Ballots from members who have died since voting	4
"Total not entitled to vote	153
"Ballots canvassed	2 933

"For Vice-President, Zone II:	
Frank O. Dufour	676
Frank L. Nicholson	252
Void	1
Blank	4
Total	933

"For Directors, District 1: (Two to be elected)	
J. P. H. Perry	479
James F. Sanborn	487
Void	1
Blank	7
Total	*506

"For Director, District 4:	
Henry J. Sherman	317
Blank	18
Total	335

"For Vice-President, Zone III:	
Frank G. Jonah	747
James L. Ferebee	310
Void	3
Total	1 060

"For Director, District 11:	
Ralph J. Reed	197

"For Director, District 14:	
W. W. Horner	174
Blank	13
Total	187

"For Director, District 15:	
Edward N. Noyes	190
Void	†47
Total	237

"Respectfully submitted,

"Lynne J. Bevan, *Chairman,*

"A. M. Brosius,
Halsey French,
Ellis E. Paul,
Joseph Farhi,
William L. Sylvester,

"Allen P. Richmond, Jr.,
T. R. Galloway,
Theodore Human, Jr.,
H. H. Pitcairn,
Raymond J. Stark,

"Andrew Kolberk,
Jacob Mechanic,
Edward W. Ritchie,
Martin C. Knabe, 3d,

"Tellers."

* Number of valid ballots cast, on most of which were two nominations for Director.

† Mr. Koch and Mr. Arneson, whose names appeared on the ballot withdrew after the ballots were prepared. Votes for them were therefore necessarily void.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from November 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

BARBARIN, MARCO PARKER, Hastings-on-Hudson, N. Y. (Age 22). Refers to H. G. Balcom, D. M. Griffith, R. A. Pendergrass.

BEICHMANN, ULRIK FREDRIK, Shanghai, China (Age 29). With E. J. Muller, Cons. Civ. Engr. Refers to H. Berents, E. J. Muller.

BERGE, FINN OLAFSSON, Shanghai, China (Age 28). With E. J. Muller, Cons. Civ. Engr. Refers to H. Berents, E. J. Muller.

BRAHDY, ALFRED, New York City (Age 45). Designing Engr., Board of Transportation. Refers to A. E. Clark, I. Jacobs, R. F. Kelker, Jr., J. H. Myers, A. I. Raisman, R. Ridgway.

BROWN, WILLIAM THURMAN, New London, Conn. (Age 28). Asst. Constr. Engr., U. S. Treasury Dept., Washington, D. C. Refers to A. C. Copland, G. K. Davis, L. F. Fairchild, F. F. Gordon, V. B. W. Poulsen, G. P. A. Stape.

BUCKLEY, JEREMIAH PAUL, Ann Arbor, Mich. (Age 23). Refers to R. A. Dodge, L. M. Gram, R. L. Morrison, C. O. Wisler, J. S. Worley.

CASHEŃ, ROBERT LEO, Hopewell, Va. (Age 28). Asst. Engr., Eng. Dept., Atmospheric Nitrogen Corporation. Refers to R. W. Burpee, A. W. French, A. J. Knight, W. R. McCann, T. C. Morris, W. Rowland, W. G. B. Thompson.

COOPER, FRANKLIN DIXON, Palo Alto, Cal. (Age 23). Acting Instructor in Civ. Eng., Stanford Univ. Refers to J. C. L. Fish, F. L. Plummer, W. E. Rice, J. B. Wells.

DAHLSTRAND, THOMAS OSBORN, Cleveland Heights, Ohio (Age 25). With Cleveland Wire Div., Gen. Elec. Co., Euclid, Ohio. Refers to W. K. Hatt, W. J. Henderson, S. C. Hollister, W. A. Knapp, G. E. Lommel, G. P. Springer.

ELIASSON, KLAS ERIK GUNNAR, New York City (Age 36). Designer with New York Edison Co. Refers to C. W. Burkland, C. B. Grady, J. G. A. Johnson, A. E. R. de Jonge, W. H. Mueser, G. A. Stenstrom.

ELLIOT, SIMON FRANK, Pueblo, Colo. (Age 23). Refers to E. O. Bergman, C. L. Eckel.

EVANS, HENRY PRITCHARD, Jr., Iowa City, Iowa (Age 20). Refers to F. J. Evans, F. M. McCullough, F. A. Nagler, H. A. Thomas, S. M. Woodward.

FERBER, FREDERICK VALENTINE, Jr., Hackensack, N. J. (Age 21). Refers to R. C. Schwind, J. C. Tracy.

FLATH, EARL HUGO, Dallas, Texas (Age 37). Dean, School of Eng., and Prof. of Elec. Eng., Southern Methodist Univ. Refers to J. H. Brillhart, O. H. Koch, W. H. Meier, E. L. Myers, E. N. Noyes, W. J. Powell, F. K. Rader.

FRIEDMAN, JOSEPH LYON, New York City (Age 20). Refers to H. W. Hayward, C. M. Spofford.

GARTRELL, FRANK LEWIS, Corpus Christi, Tex. (Age 33). Asst. Engr., Div. No. 16, Texas Highway Dept. Refers to T. W. Bailey, E. N. Gustafson, T. E. Huffman, T. C. Mitchell, D. E. Proper, A. P. Rollins, R. W. Stiles.

HARRIS, MARK LEROY, Ann Arbor, Mich. (Age 36). Senior Engr., Jensen, Bowen and Farrell and H. E. Riggs. Refers to E. B. Black, E. L. Eriksen, K. A. Farrell, H. K. Hood, O. A. R. V. Jensen, F. A. Marston, H. E. Riggs, W. C. Sadler, N. T. Veatch, Jr.

HOLLSTROM, GUNNAR EUGENE, Lawrence, Mass. (Age 24). Instructor in Civ. Eng., Worcester Polytechnic Inst. Refers to R. Fletcher, A. W. French, J. W. Howe, A. J. Knight, R. R. Marsden, C. F. Meyer.

ISELEY, ALFRED HENRY, Kalispell, Mont. (Age 33). Project Engr., State Highway Comm. Refers to C. V. Baker, C. N. Conner, T. C. Frame, H. B. Henderlite, J. B. Pridgen.

MacSWAIN, HAROLD CLARKE, East Lansing, Mich. (Age 23). Refers to C. L. Allen, C. M. Cade, C. A. Miller.

MORTON, JAMES RAYMOND, Jr., Washington, D. C. (Age 38). Designer, U. S. War Dept., Constr. Quartermaster Div. Refers to W. T. E. Barber, P. A. Blackwell, R. Coltman, Jr., O. W. Irwin, N. H. Jebejian, C. Klaesius, C. W. Ogden.

POWELL, WILLIAM LLEWELLYN, Dallas, Tex. (Age 21). Junior member of firm, Wood, Powell and Waggener. Refers to E. C. H. Bantel, E. N. Noyes.

RODERICK, RALPH EDWIN, Manhattan, Kans. (Age 24). Refers to F. F. Frazier, L. V. White.

STYLES, EDWIN BATTY, Chicago, Ill. (Age 43). Pres. and Treas., Styles Contr. Corporation. Refers to E. W. Bullard, A. B. Edwards, H. R. Gabriel, F. G. Gordon, M. Johnson, L. P. Keith, J. F. Seifried.

THURBER, PAUL, Tishomingo, Okla. (Age 28). Instructor in Eng. and Mathematics, Murray State School of Agriculture. Refers to J. F. Brookes, L. M. Bush, B. S. Myers, E. R. Stapley, N. E. Wolfard.

VAN DENBURG, JOHN WALLACE, White Plains, N. Y. (Age 43). Project Engr., Westchester County San. Sewer Comm. Refers to A. M. Brosius, L. G. Holleran, C. A. Leighton, L. G. Rice, G. A. Sherron, R. N. Whitcomb, A. M. Wyman.

WOODS, JOSEPH MARION, Edwards, Miss. (Age 35). Refers to H. A. Bestor, J. R. Hays, R. D. Likins, B. S. Philbrick, W. W. Zass.

WOOLLEY, RALPH AYERS, Springfield, Ohio (Age 25). Draftsman, Clark County, Ohio. Refers to I. N. Clover, J. P. Reinheimer, H. E. Senf, A. L. Shaw, C. H. Shea.

WORKMAN, LEWIS JACOB, Denver, Colo. (Age 30). Jun. Engr., U. S. Bureau of Reclamation. Refers to J. J. Hammond, A. B. Reeves, R. A. See, B. W. Steele, W. A. Waldorf.

WYANT, ZINT ELWIN, Jr., Topeka, Kans. (Age 23). Bridge Draftsman, Kansas Highway Comm. Refers to L. E. Conrad, O. J. Eidmann, F. W. Epps, F. F. Frazier.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

JOHNSON, WILLIAM SCOTT, Assoc. M., Jefferson City, Mo. (Elected Nov. 15, 1926). (Age 36). Chf. Public Health Engr. and Director of Div. of Public Health Eng. and Sanitation, Missouri State Board of Health. Refers to E. Boyce, C. A. Haskins, E. J. McCaustland, J. H. Porter, G. W. Putman, H. A. Rice, J. A. Strang, N. T. Veatch, Jr., C. C. Williams.

JOHNTZ, ALBERT FREDERICK, Assoc. M., Winston-Salem, N. C. (Elected Oct. 9, 1917). (Age 45). Chf. Engr., Southern Dist. Office, North-Eastern Constr. Co. Refers to E. B. Black, J. F. Brown, A. C. Everham, D. M. Garber, C. A. Haskins, W. C. Hoad, W. T. Howe, F. T. Miller, H. L. Shaner, C. W. Smedberg.

JONES, RALPH WESLEY, Assoc. M., Philadelphia, Pa. (Elected Dec. 3, 1928). (Age 35). Director of Research, Philadelphia Chamber of Commerce. Refers to C. H. Chadwick, R. L. Faithorn, L. R. Ferguson, P. L. Heslop, R. H. Horton, J. D. Justin, F. E. Lamphere, H. W. Latta, J. P. Laws, J. H. Terry, L. A. Whitsit.

MOOREHEAD, THEODORE PARKER, Assoc. M., Stockton Cal. (Elected Oct. 5, 1909). (Age 48). Structural Engr., City of Stockton, Cal. Refers to R. L. Acker, E. H. Frisell, C. Harpur, C. A. Moore, F. R. Muhs, H. L. Stevens, J. C. Stevenson.

RAPP, WILLIAM GREENFIELD, Assoc. M., Hewlett, N. Y. (Elected Junior, May 28, 1923; Assoc. M., Jan. 16, 1928). (Age 35). With McClintic-Marshall Corporation New York City. Refers to L. D. Draper E. L. Durkee, D. S. Gendell, Jr., W. A Hazard, O. J. Marsten, L. L. Martin, C. H Mercer, A. Neuffer, R. A. Pendergrass, F W. Perry, J. H. Wagner, W. H. Weiskopf

RICHARDSON, NORMAN DAYTON, Assoc M., Yonkers, N. Y. (Elected Jan. 14, 1924) (Age 44). Refers to E. A. Byrne, A. G Hayden, W. C. Lancaster, C. H. Nichols, A. I. Perry, J. H. Quimby.

SCHERMERHORN, HARVEY OBED, Assoc M., Albany, N. Y. (Elected March 1, 1910). (Age 52). Asst. Chf. Engr., Div. of Eng., Dept. of Public Works, State of New York. Refers to A. B. Cohen, F. S. Crowell, R. G. Finch, F. S. Greene, D. B. La Du, P. C. Ricketts, H. E. Smith.

STAFFORD, HARLOWE McVICKER, Assoc. M., Sacramento, Cal. (Elected Junior Oct. 10, 1916; Assoc. M. Jan. 17, 1921). (Age 40). Superv. Hydr. Engr., Div. of Water Resources, Dept. of Public Works, State of California. Refers to H. Conkling, B. A. Etcheverry, H. L. Haehl, E. Hyatt, H. D. McGlashan, T. H. Means, O. V. Stout.

VASSAR, CHARLES DAVID, Assoc. M., Brooklyn, N. Y. (Elected March 12, 1923). (Age 50). Arch. Engr., Charles B. Meyers, Archt., New York City. Refers to H. G. Balcom, P. M. Corry, J. L. Edwards, W. Vredenburgh, A. J. Wilcox.

FROM THE GRADE OF JUNIOR

CALVIN, ELMER BEN, Jun., Fredericksburg, Tex. (Elected July 15, 1929). (Age 31). Res. Engr., Texas State Highway Dept. Refers to E. P. Arneson, G. Gilchrist, T. E. Huffman, G. R. Johnson, J. T. L. McNew, J. E. Pirie, J. J. Richey.

DUDDLESTON, MORRIS THEODORE, Jun., Beatrice, Nebr. (Elected Oct. 26, 1931). (Age 32). Dist. Supt., Continental Constr. Corporation. Refers to S. E. Huey, R. O. Morrison, G. I. Rhodes, G. H. Sager, Jr., F. J. Trelease.

MAINEY, JAMES HENRY, Jun., Painesville, Ohio. (Elected Aug. 17, 1931). (Age 32). Pres. and Chf. Engr., J. H. Mainey Eng. Co. Refers to J. H. Herron, R. F. MacDowell, J. B. Morrison, F. A. Pease, W. W. Purdy.

MURPHY, GEORGE SCALLON, Jun., New York City. (Elected June 4, 1928). (Age 27). Car Engr., with Thomas F. Mullaney, Elec. Engr. and Contr. Refers to W. Beattie, J. J. Costa, F. R. Harris, J. L Harrop, A. F. Lipari, E. Praeger, J. H. Quimby.

NIXON, SAM AB, Jun., College Station, Tex. (Elected Oct. 14, 1930). (Age 32). Testing Engr., Texas Agriculture and Mechanical Coll. Refers to F. E. Giesecke, L. E. Grinter, J. M. Howe, K. E. Kirby, J. T. L. McNew, J. J. Richey, C. E. Sandstedt, J. L. Starkie, W. M. Wilson.

SADDAM, TOMAS ABRINA, Jun., Rock Island, Ill. (Elected Nov. 14, 1927). (Age 29). Asst. Hydr. Engr., U. S. Engr. Office. Refers to H. E. Babbitt, J. H. Childs, G. E. Edgerton, C. L. Hall, F. A. Nagler, G. W. Pickels, C. E. Portas.

SCHLENZ, HARRY EDWARD, Jun., Chicago, Ill. (Elected May 12, 1930). (Age 27). Sales Mgr., Pacific Flush-Tank Co Refers to H. E. Babbitt, J. J. Doland, W. L. Havens, W. C. Huntington, R. F. MacDowell, C. C. Wiley.

THATCHER, JOHN HOWARD, Jun., Brooklyn, N. Y. (Elected May 13, 1929). (Age 27). Vice-Pres., John Thatcher & Son. Refers to G. E. Beggs, T. J. Carlile, F. H. Constant, H. J. Kolb, G. A. Noren, F. N. Severud, F. N. Willson.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.

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* Director John R. Slattery died September 23, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

January 16-17, 1933:
A Quarterly Meeting will be held at New York, N. Y.

ANNUAL MEETING

January 18, 19, 20, and 21, 1933

NEW YORK, N. Y.

January 18, 1933:

Morning.—Annual Meeting. Presentation of Medals and Prizes.
Afternoon.—Technical Meeting.
Evening.—President's Reception and Dinner.

January 19, 1933:

Morning.—Technical Division Sessions.
Afternoon.—Technical Division Sessions.
Evening.—Entertainment and Smoker.

January 20, 1933:

All-Day Excursion.

January 21, 1933:

Morning.—Inspection Trips.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.